CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

AUGUST, 1952. Vol. XLVII, No. 8

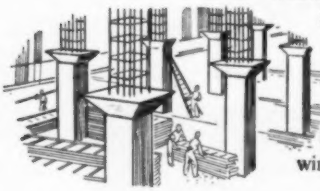
FORTY-SEVENTH YEAR OF PUBLICATION

PRICE 1s. 6d. ANNUAL SUBSCRIPTION 18s., POST FREE. $3·00 in Canada and U.S.A.

LEADING CONTENTS

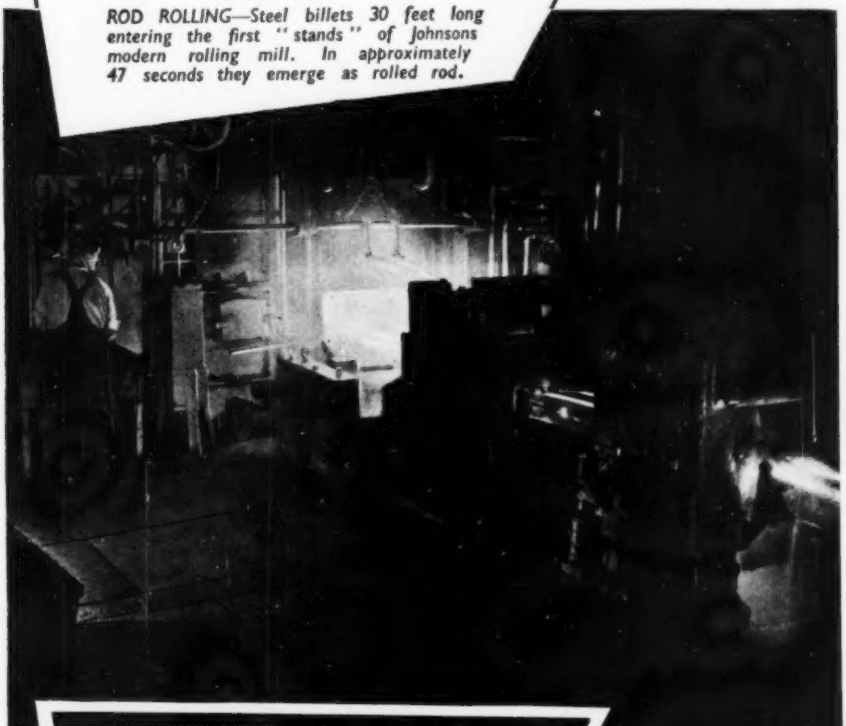
No. 537. ISSUED MONTHLY *Registered for Canadian Magazine Post*

Putting the backbone in concrete construction



Whatever the industry, there's always a place for wire. In some it's plainly visible—in others, such as reinforced and prestressed concrete work, it remains strictly behind the scenes. Yet you have confidence in its ability—you know it's there—providing a backbone—giving concrete a new strength and a new standard of safety. And when that wire is made by JOHNSONS such confidence is more than justified—it's *demanded !*

ROD ROLLING—Steel billets 30 feet long entering the first "stands" of Johnsons modern rolling mill. In approximately 47 seconds they emerge as rolled rod.



JOHNSONS WIRE . . .

FOR THE CONCRETE INDUSTRY

RICHARD JOHNSON & NEPHEW LTD., FORGE LANE, MANCHESTER 11.

COPPER STRIPS

for expansion joints

All Reinforced Concrete Engineers recognise the advantages of using copper strips for sealing joints in concrete work. Copper is ductile, will not crack under repeated bending, is non-corrosive and is unaffected by wet concrete. We specialise in the supply of perforated copper strips of all required lengths and widths for expansion joints, and shall be pleased to submit prices against detailed specification.

ALEX J. CHEETHAM LTD.

MORTON STREET · FAILSWORTH · MANCHESTER

Telephone: FAILsworth 1115/6

DS14

WASHED

BALLAST, SAND, SHINGLE &

Crushed Aggregate for Reinforced Concrete.

WILLIAM BOYER & SONS, LTD.

Sand and Ballast Specialists,

IRONGATE WHARF,

PADDINGTON BASIN, W.

MEMBERS OF B.S. & A.T.A.

DELIVERED DIRECT TO ANY CONTRACT BY MOTOR LORRY.

Quotations on Application.

Telephone: Paddington 2024 (3 lines).

BRC

FABRIC

FOR ROAD REINFORCEMENT

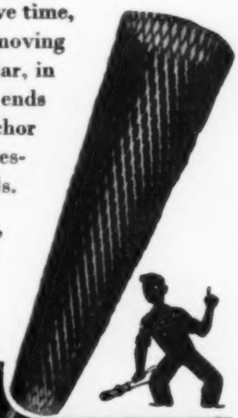
AQUAREP

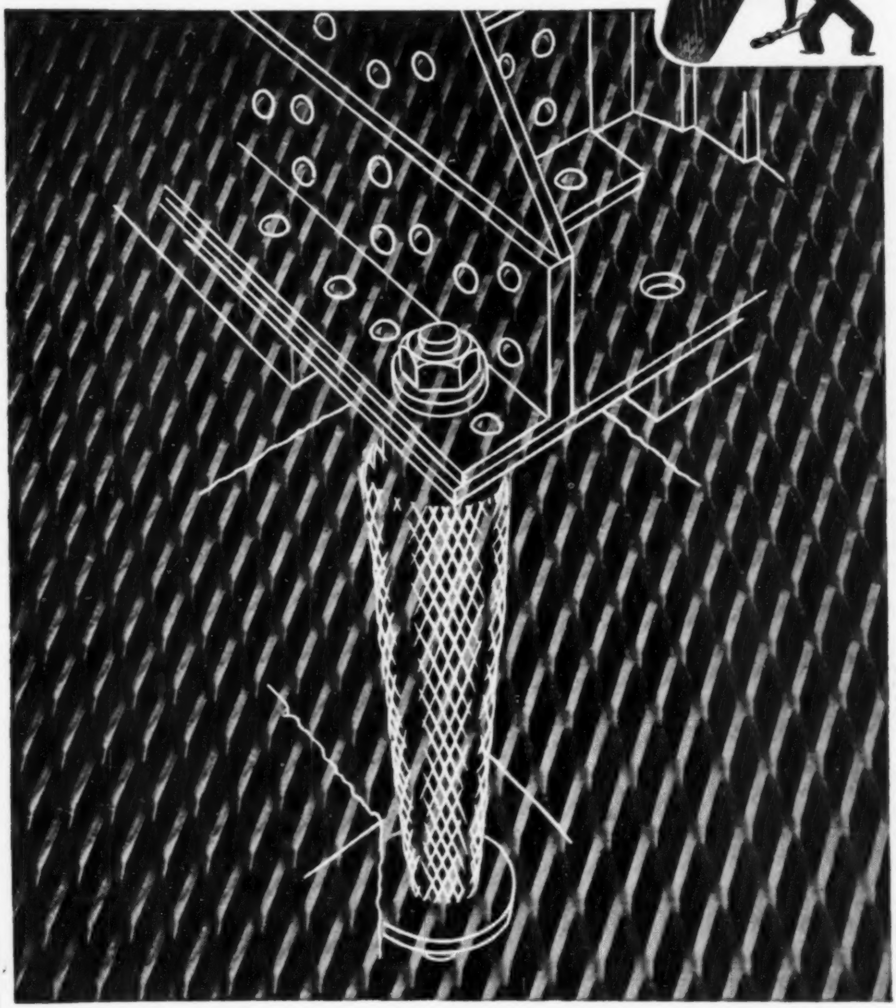
SEND FOR DETAILS

for 100% WATERPROOF CONCRETE

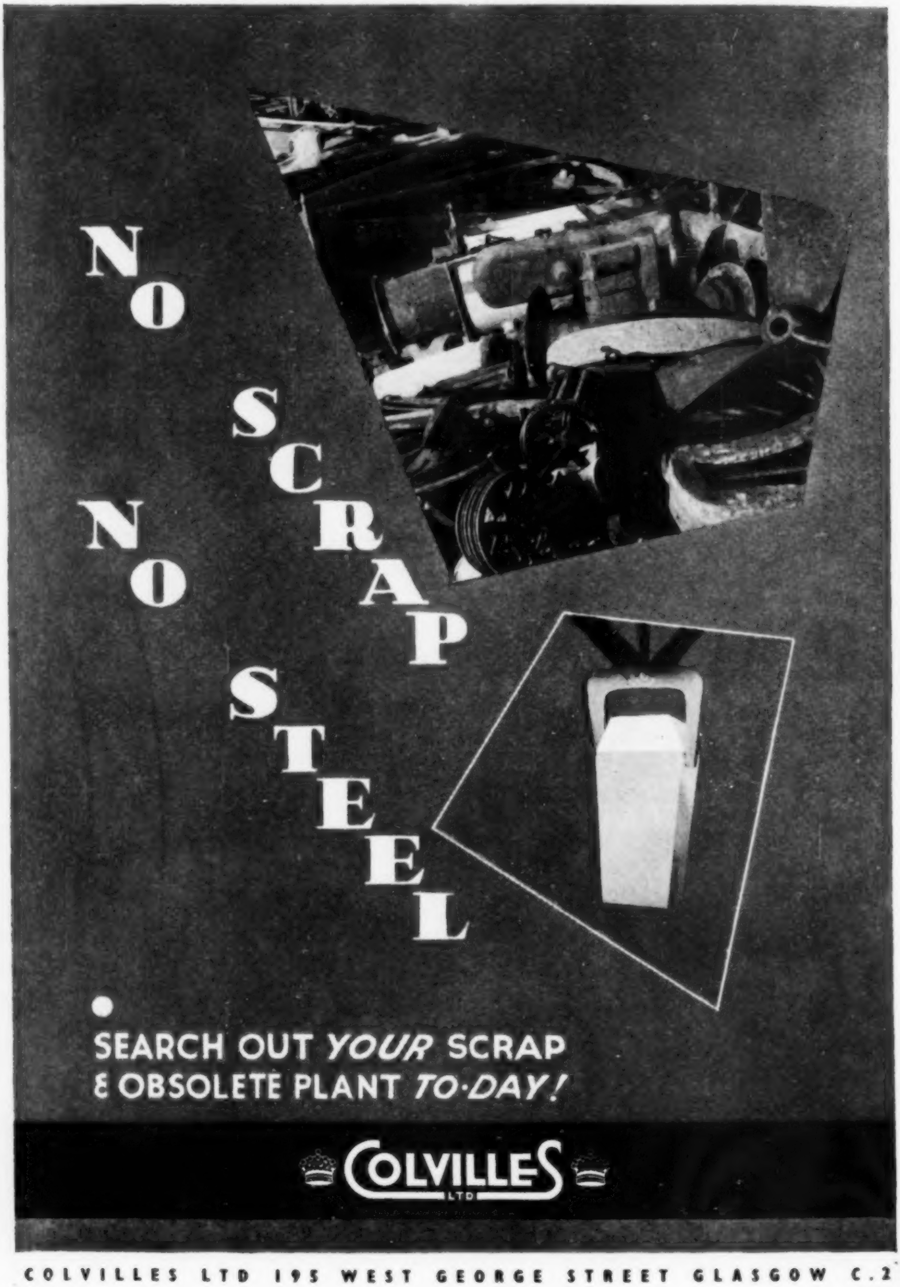
DAMP-PROOFING LTD.

DEPTFORD Tel.: TIDeway 1488-7 LONDON, S.E.






1
Prestressed Concrete (Lee-McCall System) with Macalloy Bars
An economical and effective system of prestressing concrete, using high-tensile alloy steel in bar form. The steel is provided with positive end-anchorage and does not rely upon bond to transmit the stresses to the concrete.
YOU CAN SAVE constructional STEEL
by using . . .
Write for further details
2
'Matobar' Welded Fabric Reinforcement
Economical for all types of concrete construction. Hard drawn, high-tensile steel wire mesh, electrically welded at every intersection ; permissible working stress, 27,000 lb. per sq. in. in tension.
3
Isteg Steel Reinforcement
manufactured under licence
Steel bars with a combination of twist and cold working, giving 50% improvement in tensile stress ; 30% less weight of steel. Improved bond ; hooks and overlengths eliminated.


NO SCRAP
NO STEEL
SEARCH OUT YOUR SCRAP
& OBSOLETE PLANT TO-DAY!
COLVILLES LTD


The fastest, most economical method of placing
BULK CONCRETE
giving
BETTER
placing of high quality concrete at the lowest cost per cubic yard.
BETTER
placing on awkward sites—uphill, downhill and underwater . . . traversing highways and railway tunnels without obstructing traffic.
BETTER
placing without rehandling problems . . . no chutes, hoists, towers, buckets, cranes, etc.
BETTER
utilisation of man-power—with one man more concrete can be placed than by a whole team using old-fashioned methods.
Blaw Knox concrete pipe lines on the construction of the Royal Festival Hall—Main Contractors—Messrs. Holland & Hannen and Cubitts Ltd.
PUMP IT
Basic component of the system is the BLAW KNOX portable, two speed, heavy duty, single acting horizontal piston pump. Capacity 15-20 cubic yards of concrete per hour.
PIPE IT
BLAW KNOX Concrete Pipelines comprise 6" steel pipes in various lengths up to 10' and simple, quick-acting couplings with rubber seatings for fast assembly and dismantling. During pumping the pipeline is completely filled, concrete moving through in synchronous impulses in an excellent state of uniformity.
PLACE IT
Concrete is poured straight into the forms, exactly where it is needed.
Blaw Knox engineers will be glad to advise on the application of the Concrete Pump to specific jobs.
BK.63
BLAWKNOX
BLAW KNOX Concrete by Pipeline
BLAW KNOX LIMITED, 90/94 Brompton Road, London, S.W.3
Telephone: KENsington 5151
Cables: BLAWNOX, London
Grams: BLAWNOX, SOUTHKENS, LONDON
AREA OFFICES IN BRISTOL, LEAMINGTON SPA, LIVERPOOL AND YORK. REPRESENTATION IN BELFAST, CARDIFF, DUBLIN AND GLASGOW, AND ALL PRINCIPAL CITIES OF THE WORLD

CONCRETE
AND
CONSTRUCTIONAL ENGINEERING

INCLUDING PRESTRESSED CONCRETE

Volume XLVII. No. 8. London, August, 1952

EDITORIAL NOTES

"Shell" Roofs.

Due to the large floor spaces unobstructed by columns that it makes possible, and its economy if the shuttering can be used repeatedly, the cylindrical shell, or barrel-vault, type of roof is finding increasing favour. The interest in this type of structure was demonstrated at the well-attended symposium on shell roof construction arranged by the Cement and Concrete Association and held in London last month. The symposium was timely, for so far little information has been published in this country on the design of this type of roof. The development of shell roofs in this country has been similar to that of reinforced concrete and prestressed concrete. For some years reinforced concrete was designed by a few specialists until sufficient information became available to enable any competent engineer to use the new material. For a few years prestressed concrete was designed by the representatives in this country of Continental inventors and patentees before the principles were sufficiently well known for other engineers to design prestressed concrete structures. The design of shell roofs is passing through the phase of being designed by a few specialists only, and the discussion at this symposium will give a wider knowledge of the subject which may result in more engineers designing this type of structure.

The first day was devoted to the architectural aspect of shell roofs, the next to design problems, and the third to construction. The papers on costs and methods of construction were particularly valuable for, although these roofs effect a considerable saving in concrete, this may be offset, or more than offset, by the cost of shuttering for curved concrete surfaces. It appeared to be accepted that shell roofs can be cheaper than other types of roof only if a simple type of shuttering is used which can be quickly lowered and used many times ; this problem seems to have been already solved by contractors who specialise in this class of work. It was reported that in cases where there is much repetition and travelling centres are used, and when the height above ground is small, shell roofs can be built for as little as 4*s.* per square foot of floor area, while barrels of exceptional span and height may cost as much as 20*s.* per square foot. Generally, however, the cost is between 8*s.* and 12*s.* per square foot. Variations of the span and width of barrels affect the cost, but experience has shown that a barrel whose length is twice its width is generally the most economical.

The methods described for the determination of the values and distribution of stresses were all based upon the ordinary theory of elasticity. The value of the papers lies in the comparisons made between the various solutions previously obtained and in the descriptions of methods of calculation which have, up to now, seldom been used by structural engineers. The difficulty lies in the impossibility of separating the stresses due to bending and the stresses due to direct forces ; to relate these it is necessary to express the resultants of all forces and moments at a point as derivatives of the normal displacement at that point, and finally to reduce the equilibrium equations to an equation of compatibility in which the displacement is the only dependent variable. This last is a linear differential equation of the eighth order. The differences between the solutions suggested are in the number of terms which are considered to be of negligible importance. How accurately these different theories approximate to the actual stresses in the structure it is impossible to assess, but an interesting numerical comparison was presented of the results obtained for geometrically similar shells by applying three of the more well-known methods of analysis. So far the general application of " shell " construction has been confined to semi-cylindrical barrel vaults and domes, but for many structures a shape may be found which will be more satisfactory, both to the architect and to the engineer, than these two simple shapes. The difficulty of mathematical expression of these forms has to be overcome, but with the methods of computation now available this is no longer insuperable.

Analysis has not up to now generally been followed by experimental verification of the values and distribution of the stresses computed, and the tests initiated at the Imperial College of Science and by the Cement and Concrete Association have not yet produced sufficient information to warrant any material simplification of the problem to be suggested. Very little is known, too, of the resistance of shell roofs to buckling. It may well be that the simplification of methods will lie in the theory-of-rupture methods advocated by some Continental engineers, but again these are as yet insufficiently supported by full-scale tests.

Prestressing the curved slabs and edge beams makes possible a reduction in the weight of reinforcement and economy in the cost of longer spans ; it also makes possible a considerable increase in the ratio of the length of span to the depth of construction necessary to prevent harmful deflections. In reinforced concrete it is not desirable that the span should exceed ten times the overall depth of the roof, but with prestressed concrete the diminution of the tensile stresses in the concrete so reduces the tendency for cracks to occur and the amount of vertical deflection is so much smaller that a length to depth ratio approaching 20 to 1 may be used ; these advantages are more marked in the case of north-light shell roofs. A development which may bring about considerable economy is the use of precast elements of the shell, such as the end and edge beams, with a cast-in-situ curved slab, the two being combined by the post-tensioning of cables which can be cast in the slab and passed through ducts previously formed in the precast members.

A list of the papers read at the symposium is given elsewhere in this number, and in future numbers we hope to give abstracts of some of them.

Columns Subjected to Bending in Two Directions and with Axial Loads.

By V. H. NASH-GOWER, B.Sc.

In a reinforced concrete column subjected to bending in two directions, tensile forces often act over part of the section; since reinforced concrete is considered anisotropic under such conditions, the ordinary methods of design do not apply. As an analytical solution is almost impossibly cumbersome and tedious, the following graphical method is suggested. The process is in three steps: (1) Find the direction of the neutral axis; (2) Find the position of the neutral axis and the dimension Z measured on the neutral axis as indicated in *Fig.* 1; (3) Find the values of the stresses by simple equations.

Step 1.—Draw the section ABCD shown in the figure. Find I_{xx} and I_{yy}. On the centre line EF extended draw line FH so that EF and FH are equal to I_{yy} and I_{xx} respectively, to any convenient scale; with EH as a diameter and with the centre at G, describe a circle. From point E set off EL, which is the resultant of the lines of action of the two bending moments and cuts the circle at J. The distance EL, set off to the same scale as the section, is the eccentricity, that is $\frac{\text{bending moment}}{\text{load}} = \frac{BM}{W}$. Join J to F and produce the line to meet the circle at K. Join KE; this line (KE) is the direction of the neutral axis (it is not the actual neutral axis, but merely the direction). This construction is due to R. Land, and completes the first step.

Step 2.—Redraw the section ABCD in the position A'B'C'D' so that the direction of the neutral axis is parallel to the edge of the paper. Divide the section into strips parallel to the neutral axis, the areas of the strips being C_1, C_2, C_3, etc., and find their centres of gravity.

Set up a force polygon with the line of loads $MC_1 = C_1$, $C_1C_2 = C_2$, etc. Set off the pole distance MO to any convenient scale. On the same line of loads set off $MS_1 = m$ times the area of steel S_1, $S_1, S_2 = m$ times the steel area S_2, etc. Join all the points on the line of loads to O. From the centres of gravity of the areas of concrete and steel draw lines parallel to the neutral axis, indicated as C_1, C_2, etc., S_1, S_2, etc., and draw a funicular polygon as follows. Set up a line NP at right angles to the neutral axis. Let the line C_1 intersect NP at Q. Draw the first ray QR parallel to OC_1, and carry on similarly with the other rays R, S, T, U, V.

Let the line S_1 intersect NP at W. Draw first the steel ray W—AA parallel to OS_1 to meet line S_2 at AA, and continue with the other rays AA—BB parallel to OS_2, and BB—CC parallel to OS_3. Through CC draw CC—DD parallel to the last ray OS_4 and produce beyond DD to EE where it meets the line through L' parallel to the neutral axis. Find the area of the irregular-shaped figure Q, R, S, T, DD, CC, BB, AA, W, Q by any means. Make the area DD, EE, FF, U equal to this area by swinging the line EE—FF about EE until the desired result is obtained.

Draw a line parallel to the neutral axis through FF to meet CC—EE at GG. The distance FF—GG is the required dimension Z measured to the same scale as the line of loads MC_1, C_1, C_2, etc., and this line produced upwards to cut the

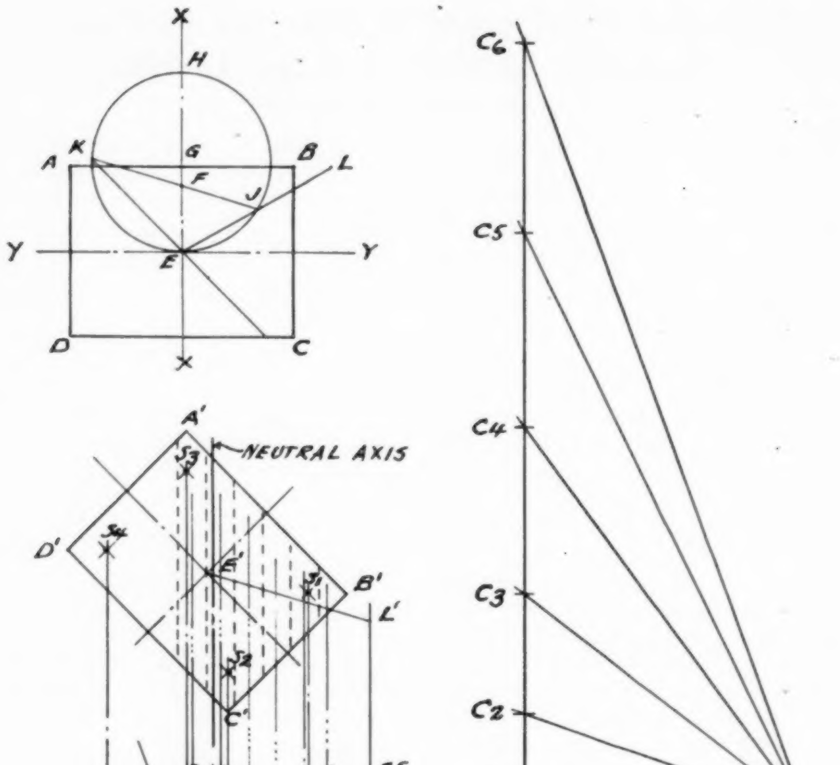

X
H
A
K
G
B
L
F
J
Y
E
Y
D
X
C
A'
S3
NEUTRAL AXIS
D'
S4
E'
S1
B'
L'
S2
C'
C6
C5
C4
C3
C2
C1
EE
S2
S3
S1
S4
FF
U
Z
DD
T
S
N
GG
R
Q
P
W
AA
BB
CC
C6
C5
C4
C3
C2
C1
M
O
S1
S2
S3
S4

Fig. 1.

section A′B′C′D′ gives the approximate position of the neutral axis in the section. For complete accuracy a small adjustment is now required, namely, that the steel areas MS_1 and S_1S_2 on the diagram should be $(m - 1)$ times the area of steel S_1 and $(m - 1)$ times the area of steel S_2 respectively, since these are on the compression side of the neutral axis. This construction is due to C. Guidi, and completes the second step.

STEP 3.—The maximum compressive stress f_c

$$= \frac{\text{Distance from neutral axis to point B}' \times \text{load } W}{\text{Polar distance OM} \times \text{Z}}.$$

The maximum tensile stress f_s

$$= \frac{\text{Distance from neutral axis to point S}_4 \times \text{load } W}{\text{Polar distance OM} \times \text{Z}}.$$

The original diagrams (shown reduced in *Fig.* 1) are roughly to scale, and the line of loads has been set out to $\frac{1}{25}$ inch = 1 sq. in.; the polar distance is set out $\frac{1}{25}$ inch = $\frac{1}{2}$ sq. in. A load of 50,000 lb. has been assumed to act at L. The rectangle has been set out at $\frac{1}{25}$ inch = $\frac{1}{2}$ inch; this gives the eccentricity as 12 in., the distance from the neutral axis to point B′ as 9·65 in. and from the neutral axis to S_4 as 7·25 in., and the dimension Z as 12·1 in., from which $f_c = \frac{9\cdot65 \times 50{,}000}{20 \times 12\cdot1} = 1995$ lb. per square inch and $f_s = \frac{7\cdot25 \times 50{,}000}{20 \times 12\cdot1} = 1500$ lb. per square inch.

The proof of R. Land's construction for the direction of the line of the neutral axis is given in German textbooks, but for the purpose of this article an original proof as follows and in *Fig.* 2, by Mr. N. Naylor, B.Sc., is given.

For any point a in a homogeneous section the stress due to bending in either plane is

$$f_{xx} = \frac{M_{xx} \times y}{I_{xx}} \text{ and } f_{yy} = \frac{M_{yy} \times x}{I_{yy}}.$$

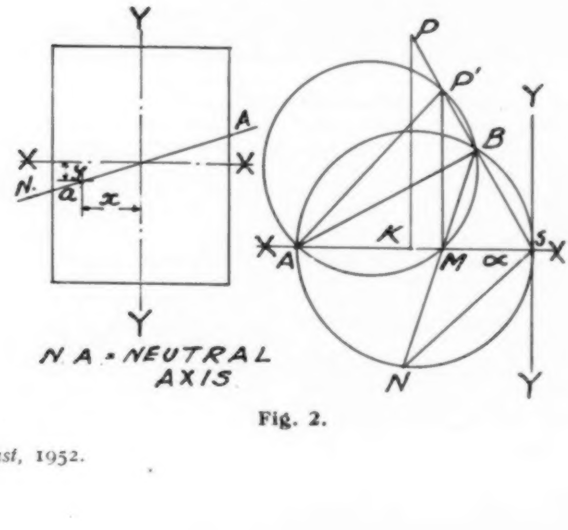


Fig. 2.

On the neutral axis these two values are equal. Therefore

$$\frac{M_{xx}}{I_{xx}} \times y = \frac{M_{yy}}{I_{yy}} \times x \text{ and } \tan \alpha = \frac{y}{x} = \frac{M_{yy}}{M_{xx}} \times \frac{I_{xx}}{I_{yy}}.$$

This gives the direction of the neutral axis.

Let XX and YY intersecting at point S be the principal axes of the section. Let P be the point of application of the load acting on the column at an eccentricity *PS*. Set out SM and MA along the XX axis proportional to I_{yy} and I_{xx} respectively. On SA as a diameter describe a circle which cuts SP in B. Join BM and produce to cut the circle in N; then NS is the direction of the neutral axis of the section.

Angle ASN equals ABN since they are both subtended at the circumference of the circle ANSB by the chord AN. Draw MP′ at right angles to AMS at M to intersect SP in P′. Since the angles ABS and AMP′ are both right angles, the points AB/BP′ are cyclic. Therefore angle ABM equals AP′M because both are subtended at the circumference of circle AMBP′ by the chord AM. Therefore the angle ASN equals AP′M.

Therefore

$$\tan \angle ASN = \tan \angle AP'M = \frac{AM}{MP'} = \frac{AM}{MS} \times \frac{MS}{MP'} = \frac{I_{xx}}{I_{yy}} \times \frac{M_{yy}}{M_{xx}},$$

which is the same result as that obtained from first principles.

$$\tan \angle ASN = \tan \angle AP'M = \frac{AM}{MP'} = \frac{AM}{MS} \times \frac{MS}{MP'} = \frac{AM}{MS} \times \frac{KS}{KP},$$

because triangles *PMS* and *PKS* are similar where *K* is part of the perpendicular from *P* and $AS = \frac{AM}{MS} \times \frac{KS \times P}{KP \times P} = \frac{I_{xx}}{I_{yy}} \times \frac{M_{yy}}{M_{xx}}$, which is the same result as that obtained from first principles.

Heat-resistant Concrete.

INVESTIGATIONS made by the General Electric Company of the U.S.A. on cements to resist temperatures up to 1200 deg. F. in connection with the testing of jet engines are described in "Engineering News-Record" for July 12, 1952.

It was found that cement that makes good heat-resistant concrete has an alumina content of at least 30 per cent. and preferably nearer to 40 per cent., a high iron content, a lime content of about 38 per cent., and no free lime. The fineness should be about 1500 sq. cm. of surface area per gramme. High-alumina cement concretes have greater heat-resistant properties than Portland cement concretes, and withstand temperatures well above 2000 deg. F.

The concrete used comprised (by dry, loose volume): One part high-alumina cement, 2·4 parts fine expanded clay, and 3 parts ¾-in. expanded clay. The water-cement ratio was 8·5 gallons per 94 lb. of cement, including all surface and absorbed water, the aggregate absorbing about 9 per cent. by weight. The mixture is equivalent to about 610 lb. of cement per cubic yard. This concrete weighs about 110 to 120 lb. per cubic foot and has a compressive strength at 24 hours (when properly cured) of 3000 lb. to 5500 lb. per square inch. Laboratory studies indicated that cyclical temperature attack might reduce the compressive strength to 2500 lb. per square inch, so a design stress of 1700 lb. per square inch was adopted. Tests on cores one year old showed strengths of 3800 to 4000 lb. per square inch and other test pieces of the same age had strengths of 2600 lb. per square inch after being subjected to high air temperatures and high internal temperatures during setting.

Prestressed Concrete Helicoidal Staircase.

THE staircase illustrated, 17 ft. 2 in. high, is at the offices in Antwerp of the General Motors Company. It is in prestressed concrete and was designed by Professor G. Magnel, who has supplied the following notes.

The design is so unusual that a full-scale model was first built in the laboratory at the University of Ghent and tested to 3½ times the working load. The dimensions of the model are given in *Fig.* 2. The design method used is not exact—exact methods do not exist in engineering—but is based on the results of the test.

Measured vertically, the thickness of the prestressed test slab was 13·4 in.; measured normal to its surface it was 10·2 in. at the inside edge and 12·6 in. at the outside edge, an average of 11·4 in. The design loads were 176 lb. per square foot for dead load and 104 lb. per square foot for live load measured on a horizontal projection, a total of 280 lb. per square foot.

Calculation gave the following maximum values: Bending moment on section A_1B_1, 141,000 ft.-lb.; torsional moment on sections A_0B_0 and A_nB_n, 211,000 ft.-lb.; shearing force on sections A_0B_0 and A_nB_n, 29,100 lb. If the vertical reactions along one of the lines of support A_0B_0 or A_nB_n are assumed to be linear (*Fig.* 3), $S_a = 28{,}100$ lb. per foot and $S_b = 37{,}500$ lb. per foot. Hence the shearing stress due to the combination of the torsional moment and the shearing force was $\frac{3 \times 37{,}500}{2 \times 11{\cdot}4 \times 12} = 411$ lb. per square inch.

Prestressing was by means of two cables each containing 16 wires of 5 mm. diameter, stressed at one end to 145,000 lb.

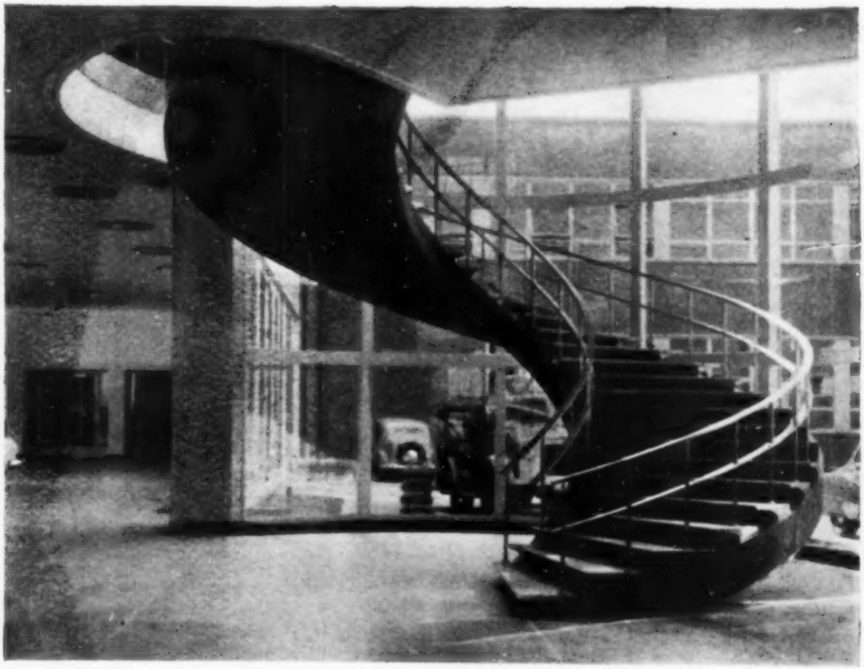


Fig. 1.—Helicoidal Staircase at Antwerp.

per square inch. This gave a total force of 141,000 lb. The loss due to friction of cables against the concrete was measured, and was 45 per cent. This left an initial prestressing force in section A_1B_1 of 77,600 lb.; and ultimately, because of a loss of 15 per cent. in course of time, the prestressing forces were

$$0{\cdot}85 \times 141{,}000 = 134{,}000 \text{ lb.}$$

in A_0B_0 and $0{\cdot}85 \times 77{,}600 = 66{,}000$ lb. in A_1B_1, producing a compressive stress in the concrete at A_0B_0 of 157 lb. per square inch.

With a shearing stress of 411 lb. per square inch and a compressive stress of 157 lb. per square inch, the maximum principal tensile stress was

$$\sqrt{411^2 + \frac{157^2}{4}} - \frac{157}{2} = 340 \text{ lb. per square inch.}$$

The stresses in A_1B_1 due to the bending moment of 141,000 ft.-lb. and the axial compression of 66,000 ft.-lb. were then computed. The cross section was $11{\cdot}4 \times 74{\cdot}8$ in., with 1·04 sq. in. of steel per foot in the tension side and 0·52 sq. in. per foot in the compression side. The stresses due to these causes were therefore 1150 lb. per square inch in the concrete and 16,600 lb. per square inch in the steel.

During the test the first crack appeared under a live load of 245 lb. per square foot. This load added to the dead load totalled 421 lb. per square foot, or 1·51 times the working load. Hence the maximum shearing stress was $411 \times 1{\cdot}51 = 620$ lb. per square inch. The maximum principal tensile stress was

$$\sqrt{620^2 + \frac{157^2}{4}} - \frac{157}{2} = 547 \text{ lb. per square inch.}$$

In section A_1B_1 there was a bending moment of $1{\cdot}51 \times 141{,}000 = 213{,}000$ ft.-lb. which, with the same prestressing force as before, produced a stress in the concrete of 1720 lb. per square inch, and in the steel of 25,500 lb. per square inch. It was obvious that the concrete did not have a tensile resistance as high as 547 lb. per square inch, although its crushing strength on 8-in. cubes was 9000 lb. per square inch. This showed that the computation was not quite correct. If the tensile strength of the concrete was assumed to be 350 lb. per square inch the calculated figure for the principal tensile

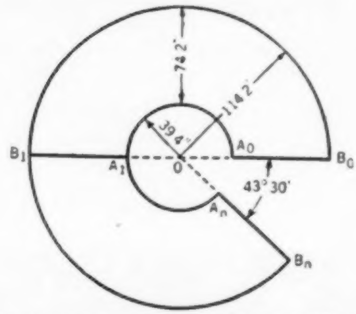


Fig. 2.—The Vertical Distance between A_0–B_0 and A_n–B_n is 17 ft. 2 in.

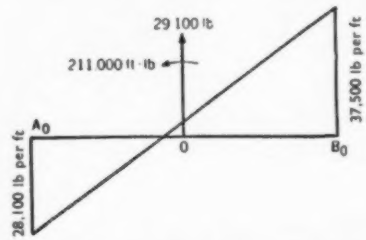


Fig. 3.—Reactions on the Line A_0–B_0 are assumed to follow a Linear Law.

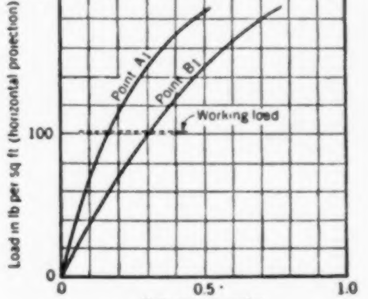


Fig. 4.—Deflections Measured at A_1 and B_1 (Fig. 1).

stress was 1·56 times too high. If the principal tensile stress due to the combination of the torsional moment and the shearing force had been computed by applying known formulæ for rectangular cross sections, values would have been

found three times higher than those given by the test. For this reason, more confidence is placed in the method used and here described.

The staircase as built differed from the model in that its width was 7 ft. 2¾ in., whereas the model had a width of 6 ft. 2¾ in. The thickness of the slab was the same, as was the percentage of steel. The prestressing was applied by six cables instead of two. This increased the applied prestressing force to

$$3 \times 141{,}000 = 423{,}000 \text{ lb.}$$

The prestressing was applied by jacks at both ends of the wires in order to reduce the friction loss from 45 to 22·5 per cent. These conditions gave the following ultimate prestressing forces: Section A_0B_0, $0{\cdot}85 \times 423{,}000 = 360{,}000$ lb.; Section A_1B_1, $0{\cdot}85 \times 0{\cdot}775 \times 360{,}000 = 237{,}000$ lb.

The loads are the same as those used for the model, but because of the increased width they gave the following values: Bending moment, 164,000 ft.-lb.; torsional moment, 245,000 ft.-lb.; maximum shearing force, 32,900 lb. per foot. Hence the value for the maximum shearing stress becomes

$$\frac{3 \times 32{,}900}{2 \times 11{\cdot}4 \times 12} = 360 \text{ lb. per square inch.}$$

The compression in A_0B_0 due to the prestress was 364 lb. per square inch. Hence the maximum principal tensile stress was

$$\sqrt{360^2 + \frac{364^2}{4}} - \frac{364}{2} = 222 \text{ lb. per square inch.}$$

Next the stresses in section A_1B_1 were computed with the bending moment of 164,000 ft.-lb. and the axial compression of 237,000 lb., and were 1120 lb. per square inch in the concrete and 9500 lb. per square inch in the steel. Again the calculated principal tensile stress was too high. Assuming the same error as for the model, this stress under working conditions would be $222 \div 1{\cdot}56 = 142$ lb. per square inch, which is low for concrete having a crushing strength of 9000 lb. per square inch.

Deflections measured during the test of

Fig. 5.—Helicoidal Staircase at Antwerp.

the model are shown in *Fig.* 4, and are reasonable under working-load conditions. The deflections of the actual staircase will be less because it is stronger than the model and also because the top of the model was not rigidly fixed. During the test care was taken to measure the loss of prestress due to the friction of the cables against the helicoidal channels in the slab. In the model, prestressing of the two cables was from one end only; in the actual staircase prestress was applied simultaneously to each pair of wires at each end of the slab.

The stair treads are of stainless steel, and the handrails are of bronze supported by posts of stainless steel. The architects were Messrs. Cole & De Roeck, of Antwerp, the consultant architects Messrs. Smith, Hinchman and Grylls, of Detroit, and the contractors S. A. Blaton-Aubert, of Brussels.

Book Reviews.

"**Strength of Materials.**" By John B. Thirlwell. (London: MacDonald & Co. (Publishers), Ltd. Price 20s.)

THIS book of 206 pages is another good treatise on elementary structural design and the strength of materials. It is written expressly for students whose objective is the passing of examinations, and it is very clearly written and well illustrated with many numerical examples. The work is entirely practical, and should also be useful to the practising engineer. Matters of particular usefulness include very clear illustrations of the application of Mohr's theorems for slope and deflection of beams and the two chapters on strain energy and the general properties of materials. There are several examples of combinations under stress of materials having varying elastic properties, a principle so necessary to be appreciated by students who may subsequently be engaged in design in reinforced concrete or other combinations of materials acting in unison.—R. P. M.

"**Acoustics in Modern Building Practice.**" By Fritz Ingerslev. (London: The Architectural Press. Price 35s.)

IT has been amply proved in recent years that the comfort of people employed in factories is an important factor in the quality and quantity of work done. The acoustical properties of a structure is one of the major problems in this connection, for the greater the reduction of noise the greater the comfort of the occupants of a building. The author of this book is the Director of the Acoustical Laboratory of the Danish Academy of Technical Sciences, and in addition to a full discussion of the science of acoustics he describes in this book many simple, and often inexpensive, methods of reducing noise in existing buildings as well as the methods of designing buildings so that there will be as little noise inside as is possible with the materials and knowledge now available.

"**The Suction of Moisture held in Soil and other Porous Materials.**" By D. Croney, J. D. Coleman, and Pamela M. Bridge. 1952. (H.M. Stationery Office. Price 2s. In the U.S.A., 50 cents.)

THIS is a Road Research Technical Paper prepared at the Road Research Laboratory of the Department of Scientific and Industrial Research, and describes laboratory methods of ascertaining the properties of suction of soils, bricks, tiles, plasters, and cements.

"**Laxton's Builders' Price Book.**" 125th revised edition. 1952. (London: Kelly's Directories, Ltd. Price 30s.)

FOLLOWING the style of previous editions, this well-known work gives prices of materials and measured work in all the building trades. To allow for fluctuations in labour rates, at the end of each section is given a percentage to add or subtract for each 1*d*. of difference in hourly wage rates.

Prestressed Precast Concrete Footbridges.

By N. A. DEWS.

An effect of the National Parks and Access to the Countryside Act will almost certainly be the reconstruction by County Councils of many footbridges that were previously the responsibility of parish councils which, chiefly for financial reasons, have been unable to give proper attention to these structures. Many such bridges are remote from good roads, so that the use of materials that can be easily moved in barrows or by tractor is an advantage ; it is also desirable to provide a type of bridge that can be quickly and easily erected, as workmen may have to be transported a considerable distance. The writer knows of cases where men were taken thirty to forty miles to a place which was still a quarter of a mile from the bridge.

The design illustrated in *Figs.* 1 and 2 is believed to fulfil the requirements for such bridges, and to provide a standard design for a reasonable range of spans. The bridge comprises a number of small precast concrete blocks of tee shape and weighing about 100 lb. each ; after assembly the structure would be prestressed with high-tensile steel bars or wires passing through the blocks and anchored at the ends.

To avoid deviation of the cable from a straight line, the bridge has a slight gradient from the centre ; this obviates the possibility of unknown frictional

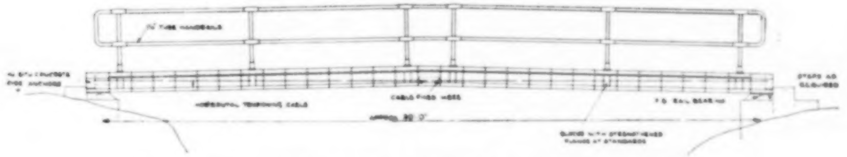


Fig. 1.—Design for a Prestressed Concrete Footbridge.

losses in the prestressing force and allows for the reduction of the required eccentricity of the cable at the critical sections to little or none at the supports where stresses due to external loads are very small. With the load for which it is designed the bridge is wholly in compression, and the blocks need not be grouted together although this might be a wise precaution to avoid damage by damp or frost. Maximum stresses under the full live load are well below 2000 lb. per square inch, which is reasonable for factory-made blocks. Shearing stresses near the supports are low and, in combination with the stresses due to prestressing, result in negligible principal tensile stresses. An approximate but conservative method of design for the end-blocks gives principal tensile stresses of less than 100 lb. per square inch across a plane inclined slightly to the horizontal and, although this is within permissible limits, a small amount of reinforcement is provided.

For erection and assembly of the bridge, which may consist of one or two lines of blocks, a light tubular-steel gantry only is required or, to span a deep gorge or swiftly-flowing stream, the bridge can be erected on the bank and swung into position (with this type of construction the bridge must be supported at the ends only, and lifting-holes are provided in the end-blocks for this purpose). It is

important that the prestressing bar or cable be positively fixed in position in the central block, otherwise the deflection of the member under incidental loads due to fixing handrailing, grouting the cable, and so on, will alter the eccentricity of the force.

The ends of the bridge rest on short pieces of small-section flat-bottom rail, and are insulated from the abutment which is made of precast blocks as shown in *Fig.* 3. Generally it is more economical to set the abutments well back from the top of the bank—an extra 5 ft. of bridge with small abutments will be much cheaper than deeper abutments which require the construction of a cofferdam.

For different spans the only calculation required is that of the prestressing force for the different dead and live loads ; the method by which this is derived is immaterial, as the combination of the final stresses is required. The method given here quickly provides a range of values of P and e for the required conditions. The notation is as follows :

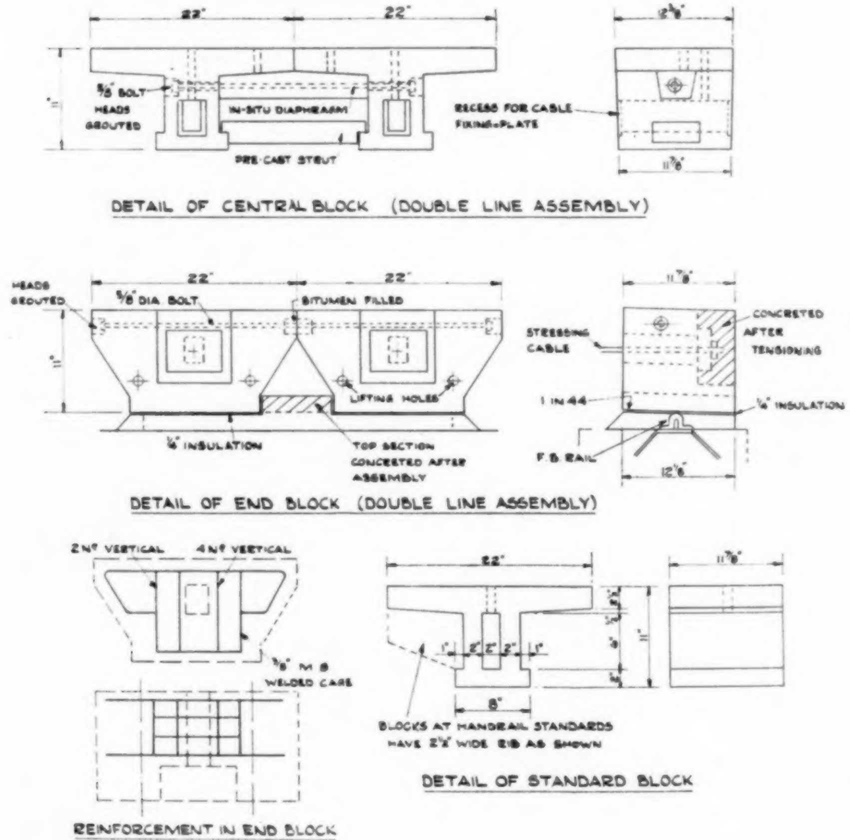
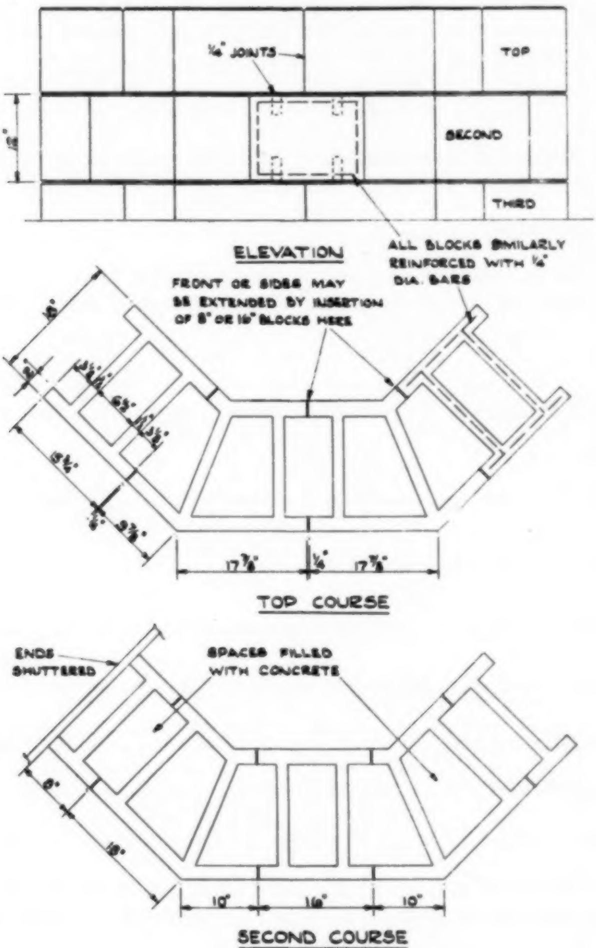


Fig. 2.

A, area of section of concrete.
P, prestressing force.
e, eccentricity of prestressing force.
y_t, y_b, distance of top or bottom from neutral axis.
r = radius of gyration.
c_t, c_b, stresses in top or bottom due to dead load.
c'_t, c'_b, ,, ,, top or bottom due to added load.
c_{pt}, c_{pb}, ,, ,, top or bottom due to prestressing force.
c_a, working stress in concrete.



ASSEMBLY OF ABUTMENT

Fig. 3.

(1) When the blocks are subjected to compression (that is "at transfer"), taking account of the stress due to the dead load only in combination with the prestressing force, the tensile stress in the top faces must not exceed the permissible tensile stress in the concrete which, in this case, is zero. In the general case of e outside the "core," that is e is greater than $\frac{r^2}{y_t}$,

$$\frac{P}{A}\left(\frac{ey_t}{r^2} - 1\right) - c_t = 0.$$

(2) After losses due to shrinkage and creep have taken place, and allowing for the stresses due to the dead and live load, the compressive stress in the top face must not exceed the allowable compressive stress c_a, that is

$$\frac{-\eta P}{A}\left(\frac{ey_t}{r^2} - 1\right) + c_t + c_t' = c_a.$$

(η is the proportion of the prestressing force available after the losses referred to.)

(3) At "transfer," and taking account of the dead load only, the compressive stress must not exceed c_a, that is $\frac{P}{A}\left(\frac{ey_b}{r^2} + 1\right) - c_b = c_a$.

(4) After losses, and allowing for the dead and live load, the stress in the bottom face must not exceed the permissible tensile stress (which, as in case 1, is zero), and $$\frac{-\eta P}{A}\left(\frac{ey_b}{r^2} + 1\right) + c_b + c_b = 0.$$

These four expressions may be rearranged in terms of $\frac{A}{P}$ and, substituting $e = 0$, become

Case 1, $\frac{-1}{c_t}$. Case 2, $\frac{\eta}{c_a - c_t - c_t'}$. Case 3, $\frac{1}{c_a + c_b}$. Case 4, $\frac{\eta}{c_b + c_b'}$.

Substituting $e = \frac{r^2}{y_t}$ in cases 1 and 2, and $e = \frac{-r^2}{y_b}$ in cases 3 and 4, $\frac{A}{P} = 0$. In *Fig.* 4 the values of $\frac{r^2}{y_t}$ and $\frac{-r^2}{y_b}$ are shown as abscissæ and values of $\frac{A}{P}$ as ordinates; values of $\frac{A}{P}$ and e, with co-ordinates intersecting within the shaded area, will then satisfy the conditions given (values of $\frac{A}{P}$ are multiplied by 1000 for convenience).

The typical section (*Fig.* 5) has the following properties (in units). Area (A), 102; y_b, 7·15; y_t, 3·85; I, 1203; r^2, 11·79; $\frac{r^2}{y_t}$, 3·06; $\frac{r^2}{y_b}$, 1·65; weight per foot, 110 lb. After assembly, and before the cable is grouted or the handrail is fixed, the bending moment for a span of 30 ft. is $\frac{1}{8} \times 110 \times 30^2 = 12{,}375$ ft.-lb. with resulting stresses of $c_t = \frac{12{,}375 \times 12 \times 3{\cdot}85}{1203} = 475$ lb. per square inch (compression) and $c_b = \frac{7{\cdot}15}{3{\cdot}85} \times 475 = 882$ lb. per square inch (tension).

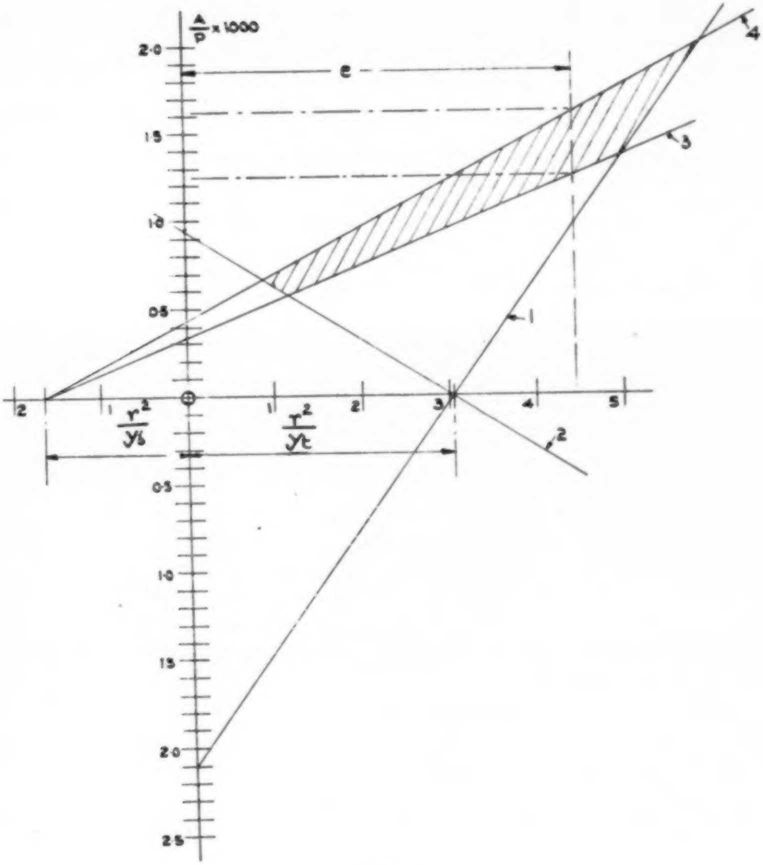

A/P x 1000
2·0
1·5
1·0
0·5
e
1
2
3
4
r²/yb
r²/yt
0·5
1·0
1·5
2·0
2·5

Fig. 4.

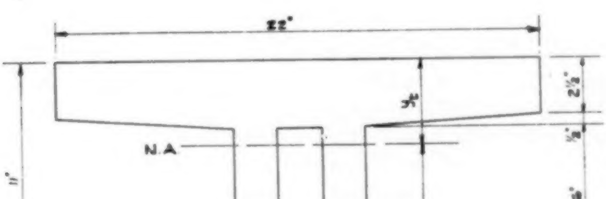

22"
N.A.
1"
2"
2"
2"
1"
8"
2½"
2"

Fig. 5.

The handrail and standards will weigh about 8 lb. per foot and the grout around the cable 12 lb. per foot, which, with a live load of 120 lb. per foot, gives a total added load of 140 lb. per foot, producing a bending moment of 15,750 foot-lb. with stresses of $c' = 605$ and $c'_b = 1123$ lb. per square inch. The values of $\frac{A}{P}$ for the four cases when $e = 0$ are plotted in *Fig.* 4 and are

$$(1)\ \frac{-1000}{475} = -2{\cdot}105. \qquad (2)\ \frac{875}{2000 - 475 - 605} = 0{\cdot}951.$$

$$(3)\ \frac{1000}{2000 + 882} = 0{\cdot}347. \qquad (4)\ \frac{875}{882 + 1123} = 0{\cdot}436.$$

(η is here assumed to be 87½ per cent., as part of the loss due to shrinkage will have already taken place at the time of prestressing; there is, however, ample margin for a greater loss without tension occurring, as will be shown.)

For the section under consideration the eccentricity can be 7·15 – 2 – half the thickness of the cable, say 4·45 in. and, referring to *Fig.* 4, it is seen that values of $\frac{A}{P} \times 1000$ may vary between 1·62 and 1·25 for this value of e or, with $A = 102$, $P = 63{,}000$ to 81,000 lb. Sixteen 0·2-in. diameter wires initially tensioned at 120,000 lb. per square inch would give a compressive force of about 60,000 lb. A 1-in. diameter bar initially tensioned at 90,000 lb. per square inch would give a compressive force of, say, 70,000 lb. The writer prefers a single bar as it makes possible a simpler end-fastening and an easier tensioning arrangement for such a bridge. Using a single bar, the stresses due to the applied force of 70,000 lb. are

$$c_{pt} = \frac{70{,}000}{102}\left(\frac{4{\cdot}45}{3{\cdot}06} - 1\right) = 312 \text{ lb. per square inch (tension);}$$

$$c_{pb} = \frac{70{,}000}{102}\left(\frac{4{\cdot}45}{1{\cdot}65} + 1\right) = 2545 \text{ lb. per square inch (compression).}$$

These stresses, after a period of time, become 273 lb. and 2227 lb. per square inch respectively (allowing for 12½ per cent. loss) and the final combined stresses are:

Top face: "At transfer," $-312 + 475 = +163$ lb. per square inch. After losses and with added load, $-273 + 475 + 605 = +807$ lb. per square inch.

Bottom face: "At transfer," $2545 - 882 = +1663$ lb. per square inch. After losses and with added load, $2227 - 882 - 1123 = +222$ lb. per square inch.

The apparent tension of 312 lb. per square inch and compression of 2545 lb. per square inch do not actually occur as the "hogging" of the member under the action of the prestressing force causes opposing stresses due to the dead load. The probable elongation of the bar under the force of 31¼ tons is

$$12 \times 31\left(\frac{90{,}000}{25 \times 10^6} + \frac{2500}{5 \times 10^6}\right) = 1\tfrac{1}{2} \text{ in.}$$

Allowing for compressive stresses 50 per cent. in excess of the working stress, the size of the end plate required, taking account of the opening in the block, is $\frac{70{,}000}{3000} + 6 =$, say, 30 sq. in., and a 6 in. × 5 in. × 1½ in. cast-steel plate is required.

A Windowless Factory in the Netherlands.

RAPID CONSTRUCTION IN REINFORCED AND PRESTRESSED CONCRETE.

A WINDOWLESS factory, the first of its kind to be built in the Netherlands, has recently been completed for Aku Rayon Industries at Emmen. The one-story building consists of four bays each 460 ft. long and 66 ft. wide, covering an area of more than 120,000 sq. ft. Together with a two-story building for the air-conditioning machinery, laboratories, canteens, and so on, the total ground-floor area of the factory is 148,000 sq. ft. From excavation to completion of the concrete work, seven months were allowed. To obtain quick construction and to meet other requirements, the foundations and the longitudinal beams of the main building are of in-situ reinforced concrete while the transverse beams are of prestressed concrete precast on the site. When work was started on the foundations a beginning was made with the casting platforms, moulds, and other preliminary work for the prestressed beams. *Fig.* 3 gives a plan of the factory and *Fig.* 2 a cross section. Precast reinforced concrete slabs laid on the upper and the lower flanges of the transverse beams form the roof and ceiling.

Another feature designed for quick construction is the uniformity of the elements of the structure. The longitudinal beams and the columns are alike, and the 232 prestressed beams are all of one type. There was a friendly rivalry between the men working on the reinforced concrete and on the prestressed concrete. The 66-ft. prestressed beams, each weighing 13 tons, were transported by rail and hauled first by hand (*Fig.* 6) and later by diesel locomotives. The actual time of hoisting a beam was only 70 seconds (*Fig.* 7), and the placing of the beams into position required a few minutes only. The beams, which are statically-determinate, were lifted at the two ends with the aid of a high two-legged hoisting apparatus held upright by steel cables (shown in *Fig.* 7). The position of the legs of the hoist had to be changed for each beam, and pile-driving gangs were employed for this work.

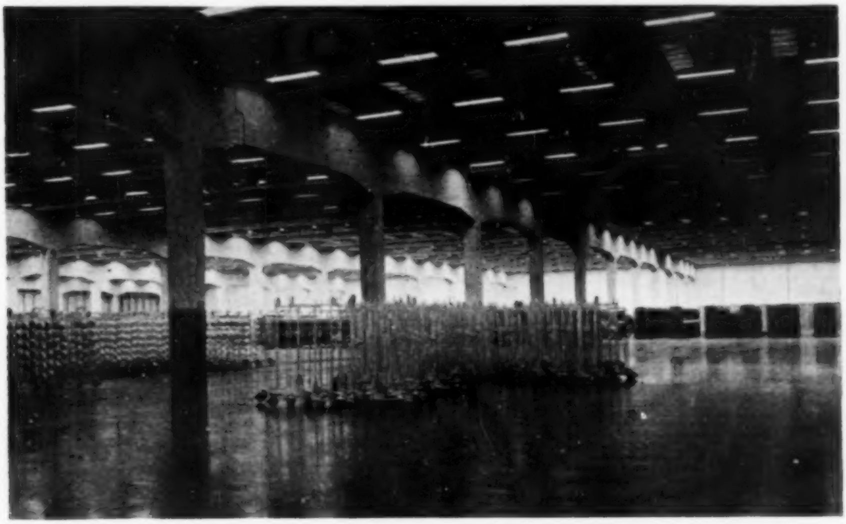


Fig. 1.—Interior of Main Building.

The prestressed concrete beams are of

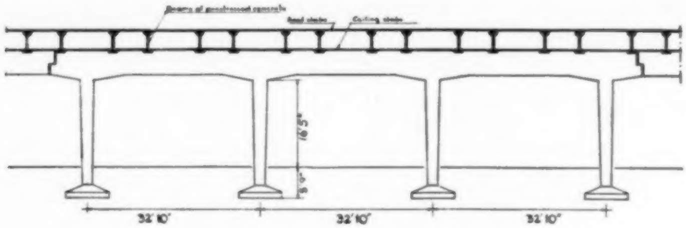


Fig. 2.—Cross Section.

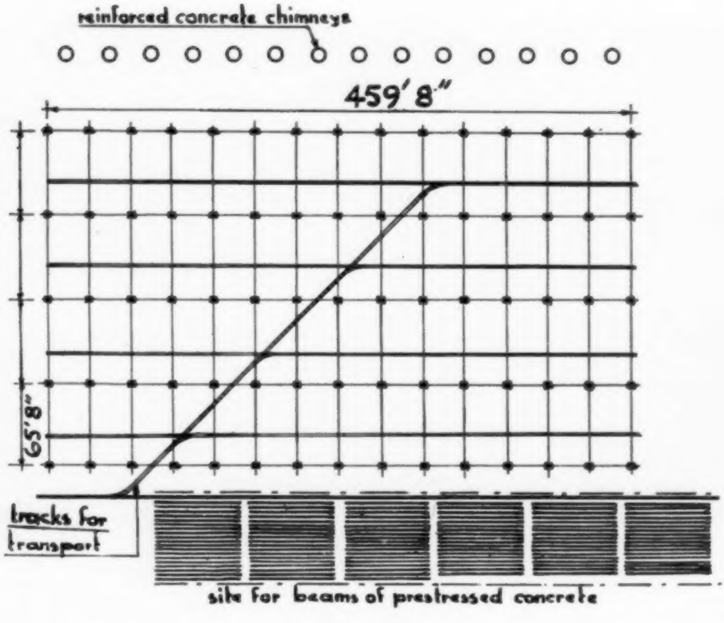


Fig. 3.—Plan.

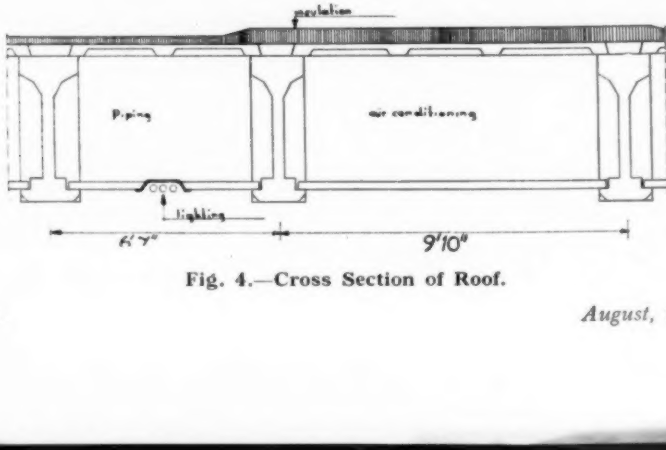


Fig. 4.—Cross Section of Roof.

I-section and have a maximum depth of 4 ft. $1\frac{1}{2}$ in. and a web $4\frac{5}{16}$ in. thick. They were prestressed after the concrete had hardened by five cables (*Fig.* 8) each of 12 wires of 5-mm. Rheinhausen hard steel in metal sheaths. The Freyssinet type of anchor was used. The primary prestressing force in each cable was 24 tons, and more than fifteen miles of cable were used.

The maximum compressive stress in the concrete when the prestressing force was

In each 460-ft. longitudinal row of reinforced concrete columns and beams two expansion joints were provided by suspending short beams between the adjacent rigid frames. To secure stability of the building, at the ends a row of precast concrete roof slabs was omitted and in place of them an in-situ reinforced concrete slab was built; together with the two adjacent prestressed beams, a horizontal beam in the plane of the roof was thus formed.

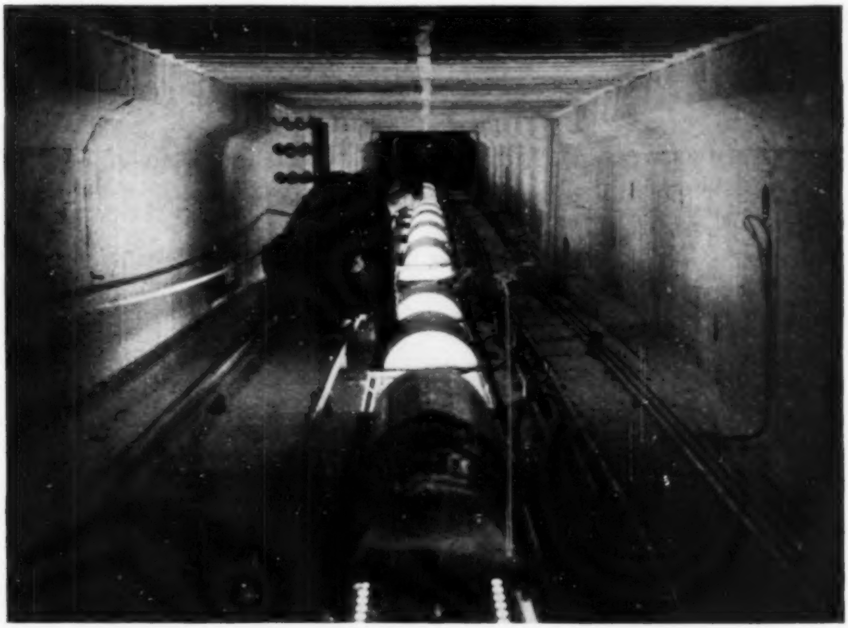


Fig. 5.—Services between Beams.

applied was 1406 lb. per square inch, and the working stress was 1193 lb. per square inch. The cube strength required at 28 days was 4260 lb. per square inch. As a cold spell was encountered and the work had to proceed, some of the beams were prestressed with two cables only when the cube strength was 2130 lb. per square inch, and when the prescribed strength in the concrete was reached the other three cables were tensioned. The building schedule required the delivery of 25 beams a week. This was later raised to 28, so that 140 cables were tensioned each week.

Between the prestressed beams and the roof and ceiling, ducts (*Figs.* 4 and 5) are formed to accommodate all the services and also the air-conditioning ducts. The prevention of dust is of great importance in rayon factories, and this arrangement offers a very effective solution.

Fig. 1 is an interior view of the factory, which is fully air-conditioned, evenly colour-conditioned, and lighted with fluorescent lamps. Instead of a vision-band which is generally advocated in windowless design, a few small windows are provided in one longitudinal wall. The floor

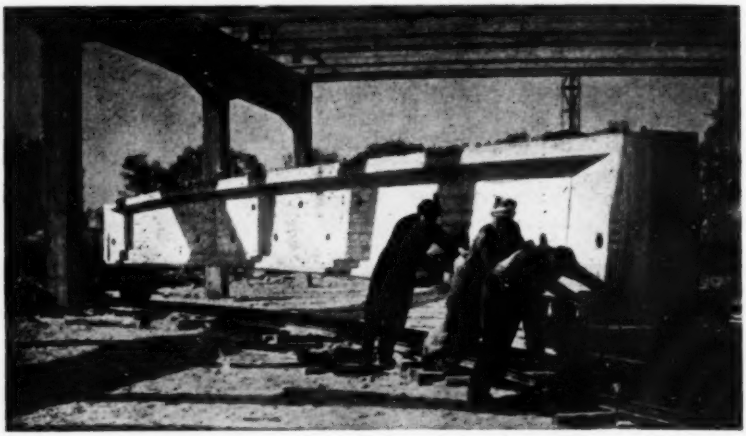


Fig. 6.—Transporting Prestressed Beams.

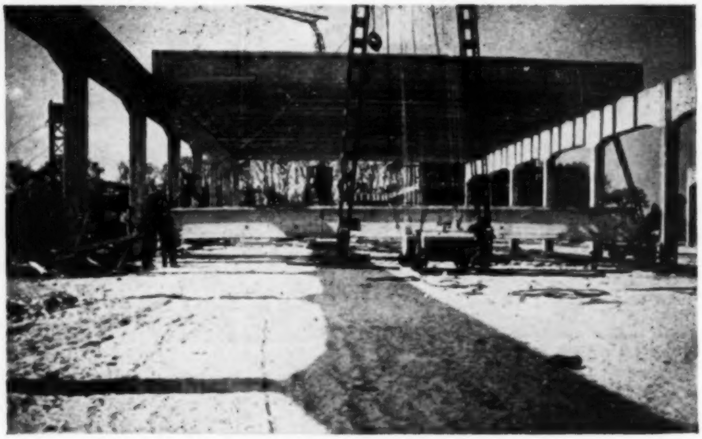


Fig. 7.—Hoisting a Prestressed Beam.

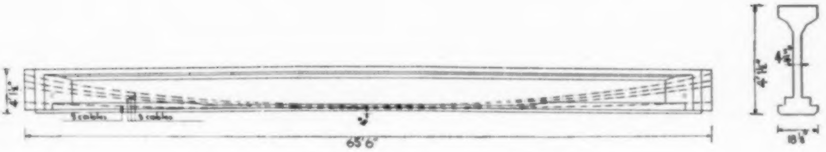


Fig. 8.—Details of Prestressed Beam.

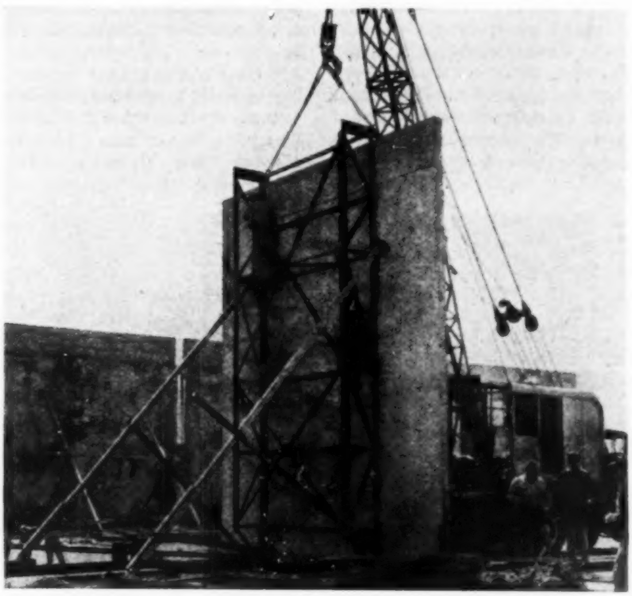
finish consists of a plastic material which is non-slippery, dust-preventing, and agreeable in colour.

No difficulties have been experienced with the workers in this windowless factory: only a few people are employed as the process is highly mechanized. The clear height of 21 ft. to the ceiling (16 ft. 5 in. under the beams) gives the impression of a large hall rather than of a factory building.

Research on the stresses under working load, on the modulus of rupture, and on the modulus of elasticity has been carried out on a full-size prestressed beam, and the results, which are in good agreement with the theory, are reported in "de Ingenieur," Nos. 4 and 7, 1952. The structure was designed by Dr. A. M. Haas, of 's-Gravenhage, in collaboration with the technical staff of Aku Rayon Industries, and erected by N.V. Nederlandsche Aanneming Maatschappij, of 's-Gravenhage.

Wall Slabs Cast Horizontally.

THE photograph (which is taken from "Concrete," U.S.A.) shows the method of constructing the walls of a single-story warehouse at Seattle measuring 220 ft. by 247 ft. on plan. The wall slabs are 6 in. thick and were cast in two sizes, namely 21 ft. square and 24 ft. square. The slabs were cast in wooden moulds on the floor of the warehouse. Fourteen days after casting the slabs were raised by a crane by means of a strong-back secured to the slabs by bolts cast in the concrete. The projecting ends of the reinforcement at the side of the slabs were tied to the reinforcement of columns 1 ft. square, which were concreted when the slabs were in position. The cost of the walls was 17 per cent. cheaper than the estimated cost of building them of in-situ reinforced concrete.



Erecting a Precast Wall Panel 21-ft. Square.

Widening Concrete Roads without Side Forms.

In the year 1952 the State of Illinois, U.S.A., is widening 950 miles of concrete roads, and it is estimated that a million dollars will be saved by the use of a method which does not require side forms. The method has been used since the year 1949, and it is stated that in the case of the work already done on several hundred miles of road the depth of the new concrete is uniform and there is little or no tendency for it to move.

In this method a trench is first excavated and the surface consolidated as shown in *Fig.* 1, and the rest of the work is done in a single operation. The machine shown in *Fig.* 2 is a converted machine used for spreading gravel. The concrete is centrally-mixed and brought to the machine by lorries from which it is emptied into a wide hopper in front of the machine. At the bottom of the hopper is a conveyor-belt operating transversely to the centre-line of the road, and which takes the concrete to a bottomless steel box on the side of the machine. This box is about 4 ft. long and from 2 ft. to 4 ft. wide according to the width of the strip being added to the road. The box slides along in the trench, serving as a temporary form for the fresh concrete. A wing-plate on the outside of the box pulls fresh soil in against the edge of the new concrete, while a pneumatic tyre, under spring-compression, compacts the soil firmly against the new concrete.

The concrete is vibrated as it is placed in the trench, usually by a vibrating bar with a power take-off from the concreting machine. A heavy steel plate, curved upward in front, slides over the fresh concrete to ensure its top surface being flush with the top of the old road. Some machines give a slightly corrugated finish to the new concrete by a steel brush attached to the rear of the machine. Curing is with a compound applied by pressure spray from a hand-propelled machine. The foregoing is from " Engineering News-Record " for April 17, 1952.

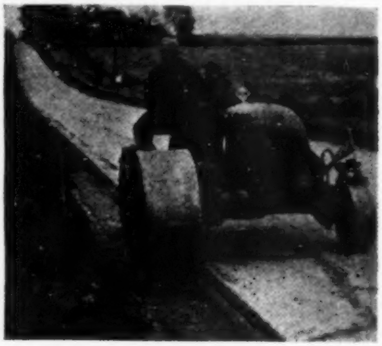


Fig. 1.—Consolidating the Base.

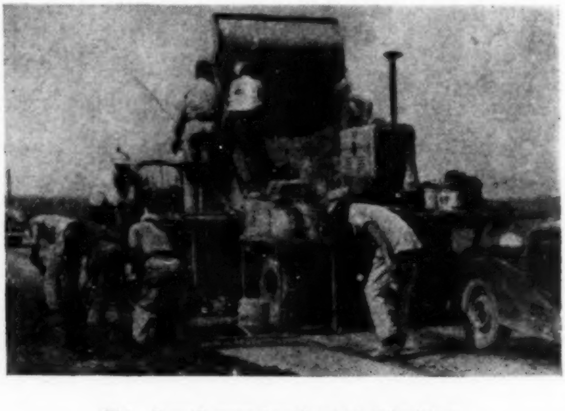


Fig. 2.—Concrete-placing Machine.

The Pathology of Reinforced Concrete.*

By HENRY LOSSIER.

CONSTRUCTIONAL DETAILS.

Silos.

The most frequent failures of silos are at the junction of the wall and floor and the cracking of the wall. Failures of the first category are rarely attributable to insufficient area of suspension bars, but more often to defective anchorage of these bars in the lower part of the cylindrical wall. It is essential that these bars should be bent-up well into the wall.

The functioning of a silo differs almost always in some degree from the simplified hypotheses made in the calculations, since this action varies with the passage of time. The only certain quantity is the weight of the contained material, unless drying occurs or a chemical change takes place. The weight of the contained material is resisted by friction against the wall and the resistance of the floor. If one of these resistances decreases the other increases and vice versa, since their sum is constant.

Now, at first the contained material is not arched and its internal angle of friction is relatively small. Its pressure against the wall is a maximum, as is the part of its weight supported by friction against the wall. The pressure on the floor is then a minimum. When, with time, the material arches, its internal angle of friction increases, the lateral pressure decreases, and the load on the floor increases. This action is sometimes further augmented by the polishing of the wall due to movement of the materials, some of which produce in addition a lubricating effect. This is the reason why it is frequently observed that cracks in the walls appear when a silo is first filled and the effect on the floor occurs more tardily.

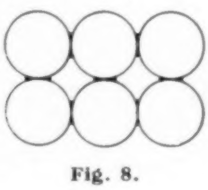


Fig. 8.

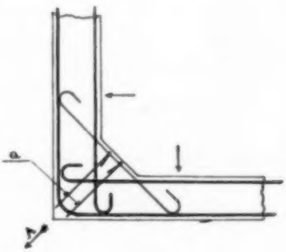


Fig. 9.

Moreover, cracks are caused by the resistance to the free play of linear variations caused by the shrinkage of the concrete on hardening and to thermal changes. This is of frequent occurrence with cylindrical containers connected together (*Fig.* 8) and which are thereby subjected to more or less intense secondary bending stresses. Finally, on filling or emptying a silo, dynamic effects, often eccentric, are produced especially at the junction of the floor and wall or at the base of a hopper-bottom, and it is necessary to increase the theoretical stresses accordingly. Consequently the best theories for the design of silos are fraught with weaknesses of some importance, and they should be modified by the results of experience.

Tanks.

Most leaks in tanks occur at the corners of rectangular tanks and at the junction of the bottom with the walls. In the first case it suffices to form at the junction of the walls well-reinforced fillets (*Fig.* 9) and to insert links (*a*) to resist the unbalanced thrust (*A*). In the second case it is necessary to take into account the secondary stresses due to the restraint which the stiffness of the bottom exerts on the free expansion of the walls, which tend to bulge at the base as shown in *Fig.* 10. There is no difficulty in calculating the theoretical value of these secondary stresses.

In the case of cylindrical tanks exposed

* Continued from July number.

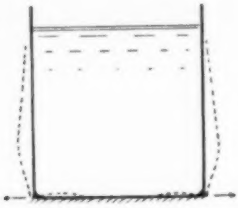


Fig. 10.

to the rays of the sun, the wall is warmed on one side only and tends to assume an elliptical form, while the bottom, being in direct contact with the ground, remains sensibly circular. This increases the tendency towards separation of the wall and floor, and ought logically to lead to an increase of the reinforcement connecting them.

Horizontal cracks are often observed in tanks which are ill-accounted for by the theoretical stresses. These cracks are generally due, at least in part, to differences in shrinkage between the wetted parts and those which are not in contact with water. Vertical bars equal to about 1 per cent. of steel should always be provided to prevent these cracks.

Crane Beams.

Beams carrying travelling cranes are often damaged by an omission to evaluate the increased loads due to shock and vibration or inadequate resistance to lateral forces. These forces are often greatly aggravated by faulty adjustment of the machinery and its consequent vibration, or by rough usage. Loads are often slung in positions not directly under the normal position of the hook, thus causing horizontal forces which are transmitted to the beams. It is always wise in designing crane beams to foresee the worst that can occur as a result of the working of the machinery and of the human element. In many cases the welding of at least some of the joints of the track-rail lessens the shocks produced by the passage of the wheels over these joints.

Foundations for Machines.

The foundations or bases of machines can often be subjected to intense dynamic effects for which allowance must be made.

For example, on the pedestals of turbo-alternators the actual loads are commonly increased by 400 per cent. to 500 per cent. due to this cause. In addition, the resonance which tends to occur when the revolutions of the machine coincide with the natural frequency of the foundation, in either the vertical or horizontal direction, can be very serious, and the determination of these frequencies is often very uncertain. On the one hand, reinforced concrete bases are of massive and complex shapes which scarcely permit of the problem being thoroughly studied, whilst on the other hand the value of the coefficient of elasticity of the concrete does not remain constant as time elapses. This is why, unless preliminary investigations are made on models truly comparable in every respect with the actual parts, it is prudent, if the results of the calculations of frequencies are favourable, to consider the possibility of resonance either at the beginning or in the course of time. As a precaution it is possible to place bars in the pedestal to permit of the addition of anchoring devices which could be fixed, in case of need, without disturbing the running of the machinery. A slight modification or addition to a pedestal usually suffices to eliminate certain resonances. Calculations of the frequencies of reinforced concrete machine bases should be regarded with reserve.

Rectangular Slabs.

Large rectangular slabs often have top cracks at the corners as shown in *Fig.* 11. These cracks are due to negative bending moments which only top diagonal reinforcement can resist, as Mesnager has demonstrated in his study of thin rectangular plates.

Unbalanced Pulls and Thrusts.

These have contributed towards some of the most serious failures. When a steel bar is bent (*Fig.* 12) an unbalanced pull is produced which tends to burst the concrete outwardly by breaking

Fig. 11.

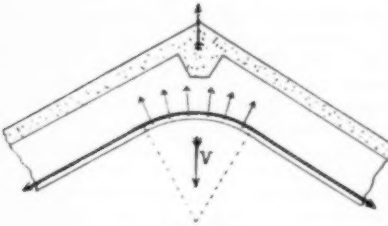


Fig. 12.

the bond of the bars and leads to fracture by bending of the member in the area affected. It follows that stirrups should be provided to resist the total unbalanced pull, and these should be close enough to prevent the bending of the parts of the bars between the stirrups. The use of crossed reinforcement bars is not necessarily always a safeguard against this danger.

A curved "shell" roof under compression is likewise subjected to unbalanced thrusts which must be counteracted by stirrups and by increasing the amount of reinforcement between the beams (*Fig.* 13). This precaution is often overlooked in the case of thin vaults with beams partly outside and partly inside the shell.

Retaining Walls.

Retaining walls are principally liable to overturning and sliding, which may occur together. The cause of these movements is often inadequate drainage. Caution should be exercised in accepting too literally results obtained by traditional methods of calculating earth pressure, particularly when the ground to be supported is not homogeneous and when it contains layers of clay. A factor of considerable importance is the investigation of the best angle of inclination for the underside of the foundation to prevent sliding.

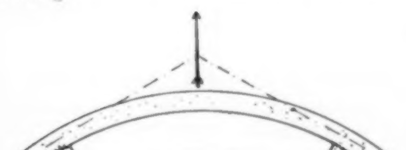


Fig. 13.

Bond Between Concrete and Steel.

The bond between concrete and steel depends on a number of factors, notably (1) Adhesion, which is generally slight; (2) A frictional effect due to the grip of the concrete on the steel caused by shrinkage during hardening; this grip is a function of the thickness of the surrounding concrete and the nearby reinforcement; (3) A keying effect produced by the roughness and inequality of the surface of the bars; it would appear that the last effect is often predominant, since truly cylindrical bars with a polished surface slide in concrete comparatively easily, and this justifies the use of milled or similar bars when high-tensile steel, which must resist high bond stresses, is used. The writer has advocated, since the year 1905, the employment of bars of this kind and of inclined stirrups welded to the main

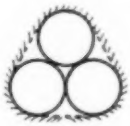


Fig. 14.

reinforcement, but the regulations of that time did not permit advantage to be taken of the resulting economy of steel.

The commonest causes of mishaps due to failure of bond are: (*a*) Main bars of large diameter being spaced too closely together and inadequately covered and (*b*) The bars touching one another; the area of each bar has been used in the calculations, although the external girth of the bundle only can properly be so considered (*Fig.* 14). The tie-bars of vaulted roofs have been known to slip at their ends through bond failure, causing partial or total collapse of the structure.

With milled or similar bars it is sound practice to increase the thickness of the cover of concrete and the number of links because of the unequal thrusts which these bars exert on the concrete owing to the loads engendering slipping. Finally, the ratio of the bond to the compressive resistance of the concrete tends to diminish with certain high-strength cements, which in addition possess other disadvantages related to shrinkage and cracking.

Joining Reinforcement Bars.

Some regulations have required that joints in reinforcement bars should be formed either by welding or by means of screwed sleeves in order to secure continuity of strength without discounting the effect of the encasing concrete. In most cases this requirement appears to be excessive, since it is possible to form adequate joints by simple lapping or splicing of the bars over a sufficient length. Three arrangements are in current use, the joints being, of course, always staggered : (*a*) The ends of the bars are hooked and lapped over a length of about 30 diameters ; (*b*) The hooks are omitted and the overlap increased ; (*c*) The ends of the bars are butted and rods added to form covers to the joints ; the last arrangement commonly used for the tensile members of large arches and bow-string girders has the advantage of having a uniform cross section without the obstructions of steel which are characteristic of the first two methods.

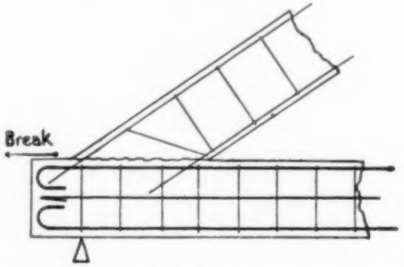


Fig. 15.

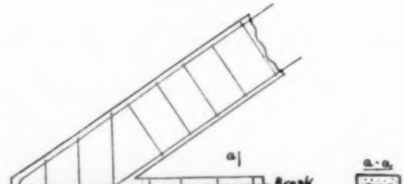


Fig. 16.

Accidents due to failure of joints generally arise from one of two causes, namely : (1) The overlaps are not long enough or are concentrated in a single zone where the excess of metal prevents the concrete developing the necessary bond ; (2) The spacing of the joints formed by butting the bars is insufficient to ensure the continuity of strength which is dependent on the bond. A typical example of case (1) is that of an arch of 150-ft. span, subjected to vibration, which collapsed suddenly after several years of service following the fracture of a tensile member, without any visible evidence, such as the formation of cracks, to give warning of the impending danger. Fractures of joints of reinforcement bars are more to be feared as they nearly always happen suddenly and without warning.

Anchoring the Ends of the Ties of Arches and Trusses.

Several serious accidents have been caused by imperfect anchorage of the ties of arches and trusses. *Figs.* 15 and 16 indicate defective arrangements which have been the cause of failures of this kind. *Fig.* 17 shows, on the other hand, the arrangement at the bridge of Lucien-Saint in Tunisia, a bow-string of 300-ft. span, which was the subject of a particularly careful study.

Shearing Stresses.

Fractures due to shearing stresses are nearly always sudden and unexpected. It is a mistake, except in slabs or other large members without construction joints, to dispense with all secondary reinforcement on the ground that the theoretical stresses in the concrete do not reach the limit of safety. There may be accidental or badly-placed construction joints, or secondary stresses due to contraction—the free play of which is restrained either by reinforcement or by other elements which the designer cannot always foresee. Ample provision to resist shearing stresses is of the greatest importance.

Effects of Shrinkage of Concrete.

The shrinkage of concrete is responsible for many minor faults but rarely for serious accidents. The spacing of contraction joints depends primarily on the features of a structure. They are often spaced at 60 ft. or 90 ft., but this distance can be much greater if flexible or hinged

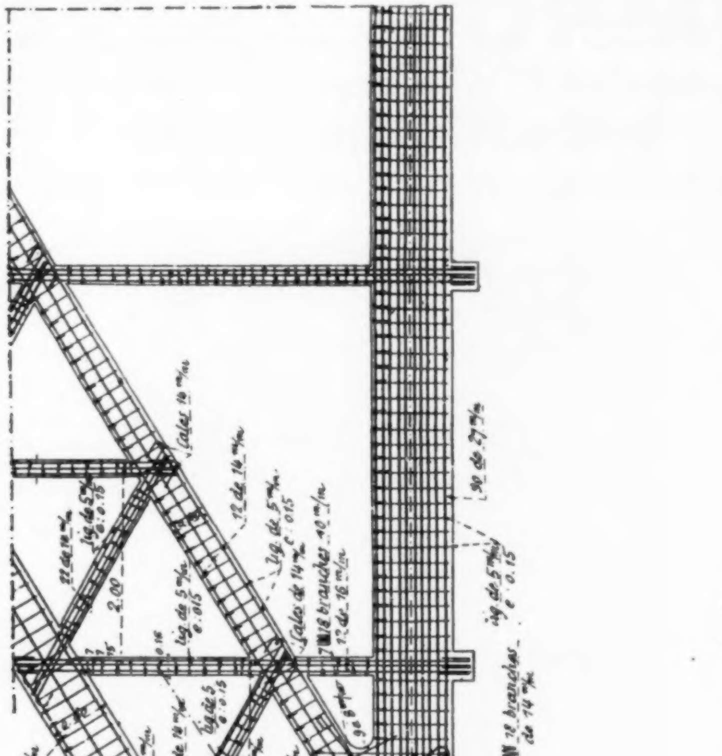


Fig. 17

columns are used, as has been done in many modern hangars. The hangar at Biscarrosse, designed by the writer in 1939, is 500 ft. long and 200 ft. wide, has a roof without a joint, and is free to expand in every direction. Similar conditions have been realised for the roofs of large tanks with a tower or self-stable pylon in the centre towards which all the expansion devices are orientated.

In monolithic structures of comparatively great depth, such as bridge girders of I or cellular cross section, cracks sometimes completely traverse the vertical web and stop close to the flanges. These cracks are more or less vertical towards the middle of the span and inclined near the supports where the shearing force is greatest. If the beams are joined by a lightly-reinforced horizontal slab, this likewise may exhibit transverse cracks; more or less inclined shearing cracks appear near the junctions of the flanges and the webs. These cracks are principally due to shrinkage, which is particularly liable to occur when cements with high initial strengths are used. Strongly-reinforced flanges restrain the free shrinkage of the webs longitudinally and these, being only lightly reinforced, normally form cracks as indicated in *Fig.* 18. In cases of this kind it is recommended that the following rules be observed so far as possible: (*a*) Avoid using cements of very high initial strength as these generally contract more than ordinary cements; (*b*) Avoid sudden changes in the percentage of reinforcement, and the use of large diameter bars in members of relatively small dimensions; (*c*) Avoid proportions of steel less than 0·50 per cent. and, if possible, of 1 per cent. in members liable to be in tension, even if the reinforcement placed elsewhere can resist the whole bending moment. Reinforcement does not always ensure immunity from cracking, but it reduces the width of cracks and in consequence the liability of the steel to corrode even if the number of cracks is increased.

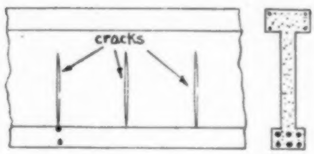


Fig. 18.

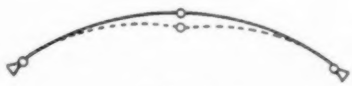


Fig. 19.

Summing up, if precautions against shrinkage are not taken, defects result consisting principally of cracks, slipping of the bars, and lack of watertightness which, whilst being objectionable, do not impair the stability of a structure. A crack, however, causes anxiety to the owner of a structure.

Creep of Concrete.

The process known as creep of concrete is not fully understood. It has caused difficulty in the use of long-span hangars closed by sliding doors; as the clearance for the doors was not sufficient to allow the increased deflection of the trusses as time went on, jamming resulted. It is now customary to allow for this effect, and this type of fault has been almost eliminated. Similar provision in the form of an initial camber must be made in bridges, especially arches and bow-string girders. A few arches may be seen which, on account of creep, have gradually developed a sag at the crown (*Fig.* 19). Secondary stresses have been caused by creep in some structures dependent for stability on their shape, and in at least one instance a ribless dome has collapsed as a result of creep.

(*To be continued.*)

Transporting Concrete by Cranes.

MOBILE tower-cranes are used for transporting concrete on French hydro-electric works for some of the dams and power stations up to 250 ft. high. The cranes are generally of two sizes, namely 45 tons-metres and 90 tons-metres, and are provided with buckets containing respectively about $\frac{5}{8}$ cu. yd. and $1\frac{1}{4}$ cu. yd. of concrete. A typical installation is that used at the dam (*Fig.* 1) constructed by the firms of Truchetet et Tansini and J. Pascel et Fils, at Aussois in the Haute Maurienne, where three tower-cranes were provided on a track laid at the foot of the downstream face of the dam. Skips of concrete were brought within reach of the cranes on a wooden gantry.

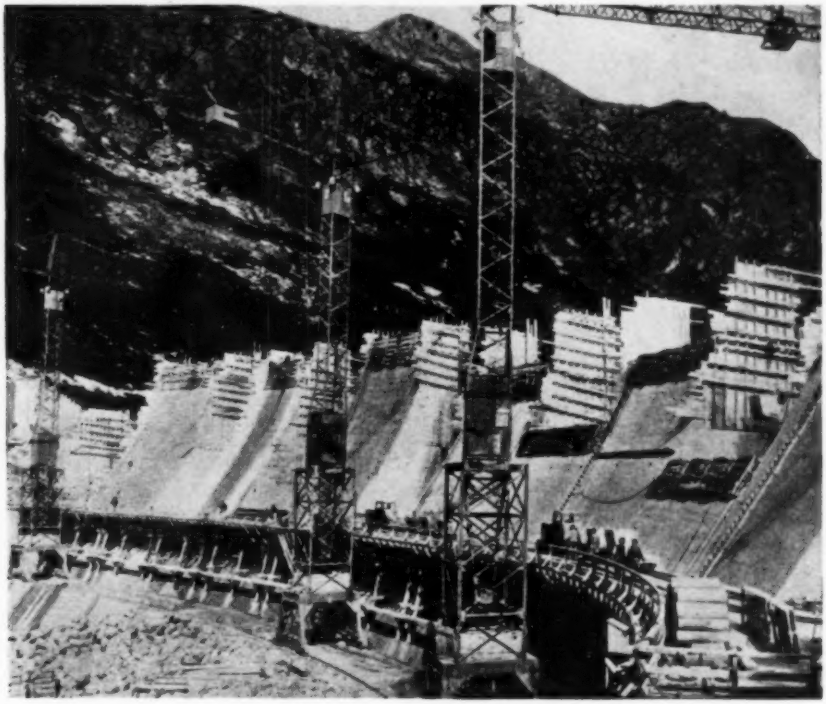


Fig. 1.—Cranes at Aussois Dam.

The lower part of the dam at Bort-les-Orgues, which is nearly 400 ft. high and forms part of the harnessing of La Dordogne river in the Massif Centrale, was concreted from bridge-type gantry-cranes (*Fig.* 2) and the upper part by tower-cranes (*Fig.* 3). Details of the equipment for the two main phases of the construction are shown in *Fig.* 4. (The illustrations are from "Travaux".) A steel gantry about 270 ft. high and 165 ft. wide was erected to straddle the site. The columns A were on the outside of the upstream face L of the dam, and were therefore entirely recoverable, but the lower parts of the columns C and some of the braces B were not recoverable as they are behind the downstream face M and are embedded in the concrete. In the first phase, skips containing about 4 cu. yd. of concrete from the central mixing station travelled, suspended from a runway D on the downstream face at the

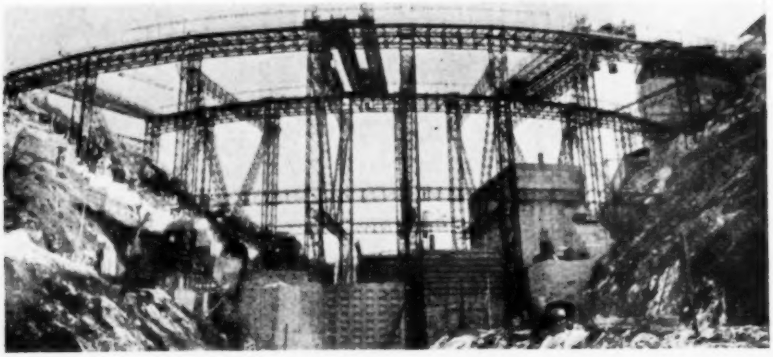


Fig. 2.—Bort Dam : Gantry for First Phase of Construction (January, 1950).

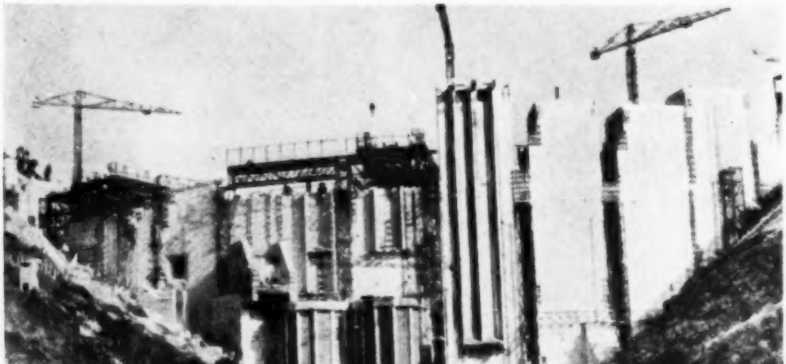


Fig. 3.—Bort Dam : Upstream Face during Second Phase of Construction (October, 1950).

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Box number 1s. extra. The engagement of persons answering these advertisements is subject to the Notification of Vacancies Order, 1952.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATION VACANT. The Reinforced Concrete Association invites applications for the appointment of Technical Assistant. Candidates must be Corporate Members of the Inst.C.E., not less than 30 years of age, with a good general knowledge of reinforced concrete design and construction. Languages an asset. Salary not less than £900 per annum. Applications, which will be handled in strict confidence by the Secretary until after a preliminary interview, to be addressed to him at 94–98 Petty France, London, S.W.1, marked "Technical Assistant."

SITUATIONS VACANT. Experienced reinforced concrete designers and detailers required for foundations and superstructures of chemical plant and oil refineries. Also vacancies for younger men wanting experience of large projects. Salary according to capabilities. Generous pension scheme, and free life insurance. Write CHIEF ENGINEER, E. B. BADGER & SONS (G.B.), LTD., 40 Parkgate Road, London, S.W.11.

SITUATIONS VACANT. Reinforced concrete consulting engineers, Westminster, have two vacancies for designer-detailers. Salaries £650 and £750 according to experience. Applications to Box 2560, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Designer-draughtsman required for detailing precast suspended flooring. Knowledge of reconstructed stone detailing also an advantage but not essential. KINGSTON CONCRETE PRODUCTS, LTD., Ryde Avenue, Hull.

SITUATIONS VACANT. Consulting engineers, Westminster, require reinforced concrete detailers. Write Box 2571, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Design engineer, bridges and buildings, prestressed concrete, and/or reinforced concrete, required by Victoria Street consultants. Degree and good previous experience essential. Progressive. Alternate Saturdays off; holiday subject to arrangement. Applications, confidential, British subjects only, to Box 2572, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Senior and junior civil engineering draughtsmen-detailers required for steelwork, reinforced concrete, and building work for electricity generating stations and general industrial work. The salary will be not less than £700 per annum for senior appointments, and will be under constant review. Full superannuation scheme in operation. Applications, giving full details of experience, etc., to C. S. ALLOTT & SON, CONSULTING CIVIL ENGINEERS, 1 North Parade, Manchester, 3.

SITUATION VACANT. Consulting engineers require structural draughtsman with experience of structural steelwork and/or reinforced concrete. Knowledge of design desirable. Varied type of work. Write giving details of experience, and salary required. HOARE, LEA & PARTNERS, 52 Green Street, London, W.1.

SITUATIONS VACANT. The British Reinforced Concrete Engineering Co., Ltd., require several qualified designers with specialist experience for their Stafford, London, Bristol, Glasgow, and Newcastle-upon-Tyne offices. Five-day week, and staff pension scheme. Apply to B.R.C. ENGINEERING CO., LTD., Stafford.

SITUATIONS VACANT. The Trussed Concrete Steel Co. Ltd., Truscon House, 35–41 Lower Marsh, London, S.E.1, require additional staff in their London and Manchester design offices: (*a*) Designer-Detailers (age 25 or over), (*b*) Detailer-Draughtsmen (age 20 or over). Previous good experience in reinforced concrete drawing office is important. Five-day week and pension scheme. Apply in writing to the above address giving full particulars of age, education, and previous employment.

SITUATIONS VACANT. Structural engineering designer/draughtsmen required in Designs Branch by Air Ministry Works Department. Applicants should have had several years' experience in the design and detailing of reinforced concrete or structural steelwork. The appointments will normally be in London. Salaries are in ranges up to £733 per annum with starting pay dependent upon age, qualifications, and experience. Overtime and extra duty allowance payable in accordance with standing regulations. Applications, quoting Code No. AP, and stating age, qualifications and previous appointments (with dates), should be sent to the nearest local Employment Exchange marked Order No. Borough 686.

SITUATIONS VACANT. Structural draughtsmen with experience in reinforced concrete detailing required in London office of NORMAN & DAWBARN, Architects and Consulting Engineers, 5 Gower Street, London, W.C.1. Reply stating age, experience, and salary required.

SITUATIONS VACANT. Structural designer-detailers wanted for interesting work full or part-time in professional engineer's office, North London. Box 2576, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Several reinforced concrete detailer-draughtsmen required. Five-days' week. Pension Scheme. C. J. PELL & PARTNERS, CONSULTING ENGINEERS, 4 Manchester Square, London, W.1.

SITUATION VACANT. Senior civil engineering draughtsman required in a large London concern. Candidates should have recent practical experience in the design, construction, and detailing of reinforced concrete industrial buildings, structures, and foundations, and some knowledge of structural steelwork design. A good knowledge of structural theory is essential. Write stating age, full details of qualifications and previous experience (with dates), and salary required, quoting this paper, to Box 2575, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1. Original testimonials should not be forwarded. Closing date 23 August, 1952.

SITUATION VACANT. Firm of civil engineering contractors desire to contact experienced and qualified engineer with connections, and able to invest moderate capital jointly with advertisers in new company to be formed. Reply in first instance in confidence to Box 2574, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

WANTED.

WANTED. Romney hut, standard width 35 ft., approximately 80/90 ft. long. Price, condition, and where to view to Box 2573, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

EXCHANGE.

REQUIRED. Black mild steel round reinforcing rods of $\frac{3}{8}$ in., $\frac{1}{2}$ in., and $\frac{5}{8}$ in. diameters in exchange for $1\frac{1}{4}$-in. diameter round, and $\frac{7}{8}$-in. diameter Isteg (equivalent to $1\frac{3}{16}$-in. diameter round). Apply RIBAR, LTD., Higher Swan Lane, Bolton.

FOR SALE.

REINFORCEMENT: 350 tons barbed wire, concertina type, in 40-lb. coils, at £21 per ton; 10 tons mixed round hexagons at £32 10s. per ton. STEEL PLATES: 10 tons $\frac{3}{16}$-in. to $\frac{3}{8}$-in. plates, sizes 2 ft. × 9 in. to 15 ft. × 24 in., at £30 per ton. STEEL SHEETS: 20 tons $\frac{1}{8}$ in. painted, non-bending quality, minimum size 3 ft. × 2 ft., at £32 10s. per ton. STEEL ANGLES: 25 tons 5 in. × 5 in. × $\frac{3}{8}$ in. × 18 ft. long at £26 per ton. SHEARED STEEL STRIPS & PLATES: Supplied in all widths and gauges. FABRICATION & PRESSWORK of all kinds; special angles, channels, gutters, etc. E. STEPHENS & SON, LTD., 58/64, Bath Street, London, E.C.1.

FOR SALE. Pneumatic pick compressor with 3 take-off points for sale. Armstrong-Whitworth with Dorman petrol engine, 100 c.f.m. at 100 p.s.i. Dennis chassis, enclosed cab. Photograph from F. J. EDWARDS, LTD., 359 Euston Road, London, N.W.1. EUSton 4681.

top of the gantry, and were picked up by traversing lifting-gear of which there were two on each of the two travelling bridges E. The lateral movement of the traversing lifting-gear and the longitudinal movement of the bridge allowed concrete to be deposited at any point below level N. Empty skips were returned to the mixing station on the runway F.

For the second phase, that is for placing concrete above level N, the steel members A and B, the two bridges E, and the runway F were dismantled, and four 90 tons-metres tower cranes H were erected on a track K on a step on the downstream face at level N. While the change-over in the plant was being made concreting was not stopped, as the skips arriving on runway D could be returned empty on runway J. In the second phase, skips containing about 2 cu. yd. of concrete arrived on runway D and their contents were discharged into other skips G and returned on runway J. The skips G were transferred by the cranes to the place where the concrete was deposited. The dam was constructed by Entreprises Metropolitaines et Coloniales.

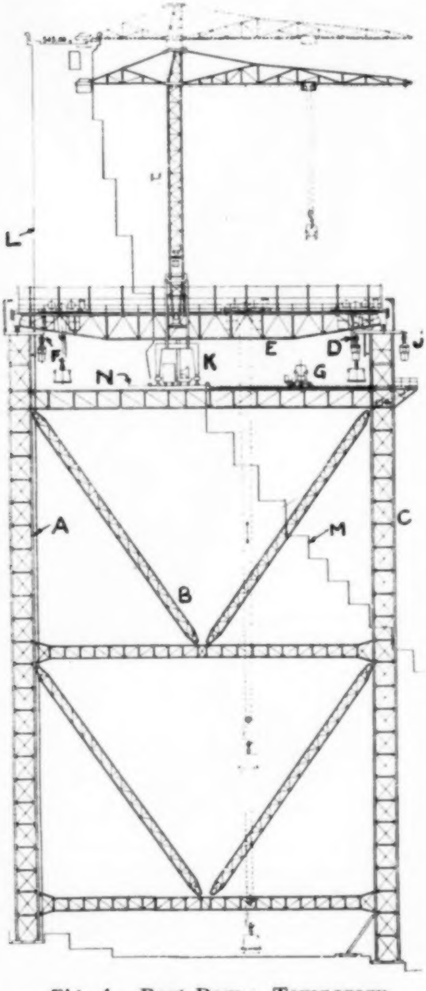


Fig. 4.—Bort Dam: Temporary Concreting Gantry.

Standard for Steel Reinforcement.

An amendment of British Standard No. 785, "Rolled Steel Bars and Hard Drawn Steel Wire for Concrete Reinforcement" includes the following revisions. Page 5, Part 2, Clause 2: For high-tensile steel the limits for sulphur and phosphorus are each increased to 0·06 per cent. maximum. Page 9, Part 3, Clause 17: In the first line "elongation" is changed to "proof stress". Page 10, Clause 18: The requirements regarding elongation are deleted and the following is added—The 0·25 per cent. proof stress, determined as described in Appendix C, shall be not less than 70,000 lb. per square inch. Page 10, Clause 19: The following is added—One test for proof stress shall be made from every 50 or part of 50 coils or, where wire is cut into lengths, from every 50 or part of 50 bundles. An appendix on the Determination of Proof Stress is added.

A Symposium on Shell Roofs.

The following is a list of the papers read at the Symposium on Shell Roof Construction arranged by the Cement and Concrete Association and held at the Institution of Civil Engineers, London on July 8–10, and which is referred to in our Editorial note. Domes, Vaults, and the development of Shell Roofing, by Mr. Leo M. De Syllas; Various Forms of Shell Roofing and their Application, by Mr. Edward D. Mills; Architectural Problems of Shell Roofing, by Mr. E. Leslie Gale; Existing Methods for the Analysis of Concrete Shell Roofs, by Dr. J. J. McNamee; Flexibility Coefficient Methods and their Application to Shell Design, by Mr. A. Goldstein; Research on Shells, by Dr. P. B. Morice; The Theory of New Forms of Shell, by Mr. R. S. Jenkins; The Combination of Shells and Prestressing, by Mr. C. V. Blumfield; Design and Construction from the Economic Aspect, by Mr. H. G. Cousins; Construction of Skelton Grange Power Station at Leeds and a Factory at King's Lynn, by Mr. H. E. Manning; Formwork used on a Factory at Greenford, by Mr. H. F. Rosevear; Construction of Self-supporting Reinforced Concrete Vaults at Antwerp, by M. C. Wets and M. A. Paduart.

Published by Concrete Publications Ltd., 14 Dartmouth Street, London, S.W.1. Telephone: Whitehall 4581
Printed by Butler & Tanner Ltd., The Selwood Printing Works, Frome and London